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The ACI JOURNAL technical pages are a medium for reporting what is of current interest in the concrete field; in research, design, construction, maintenance, or manufacturing. Members and readers are urged to "have their say" as a means of sharing knowledge and experience; time and changing interests demand that new names and new ideas take their places beside the old. Contributions describing new materials and techniques or new applications of familiar practices are welcomed by the Institute.

To guide the flow of material the Board of Direction has assigned to the Technical Activities Committee the selection of papers, committee reports, discussions, and other contributions for JOURNAL publication. To assist in their technical appraisal a corps of experts in various segments of the field volunteer their services. From the technical standpoint, acceptable contributions present original material for improvement of design, manufacturing, or construction; confirm, revise, or upset established ideas or practices; or review, digest, or arrange accumulated experience for more ready use.

General considerations by TAC are whether the contribution will be of interest and service, will fit into a well-balanced publications schedule, and can be published at reasonable expense to the Institute.

To expedite appraisal and publication, if accepted, contributors are requested to furnish glossy prints or inked tracings of illustrations accompanying manuscripts submitted for consideration. Further details of form and physical arrangement of contributions begin on p. 14, 1953 *ACI Directory*. Separate prints of the "publications policy" are available free upon request to the Secretary.

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In ACI Journal next month

In "Structural Applications of Hyperbolic Paraboloidical Shells," FELIX CANDELA presents a brief review of theoretical considerations, formulas, and applications of hyperbolic paraboloid surfaces. Their stress conditions can be pictured by membrane stress equations. Bending stresses occur only in particular cases and are limited to small zones near supports. Because they are skewed or nondevelopable surfaces, they resist buckling. These features allow a minimum thickness and the elimination of stiffening ribs.

"Shear Strength of Reinforced Concrete. Part 2—Tests of Restrained Beams Without Web Reinforcement," by K. G. MOODY, I. M. VIEST, R. C. ELSTNER, and E. HOGNESTAD, is the second of a series beginning in this issue. Data are presented on the shear strength of 61 restrained beams without web reinforcement. Tests were carried out in five series with the following variables: (1) concrete strength and percentage of longitudinal reinforcement, (2) beam depth, (3) ratio of shear span to effective depth of beam, (4) cutting off the reinforcement in accordance with the ACI Building Code, and (5) relative magnitude of negative and positive moments.

A study on a "Simplified Method for the Determination of Apparent Surface Area of Concrete Products" is presented by L. F. GLEYSTEEN and G. L. KALOUSEK. The hypothesis that the cementitious solid of autoclaved concrete products is crystalline and that of moist-air or atmospheric-steam cured units is gel-like in form served as the basis for this study. Estimate of degree of crystallinity was based on surface areas computed from nitrogen adsorption data. The results contradicted the hypothesis.

MILO S. KETCHUM describes the "Design and Construction of a Folded Plate Roof Structure." The structure utilizes two types of folded plate shells: a discontinuous Z-shaped shell arranged to provide a north light clerestory and a continuous series of inverted V-shapes. The latter shell is designed to carry two 15-ton cranes on the roof system. The analysis of stresses was made using the latest refinements involving the effect of deflection on the transverse stresses in the slabs.

"Ribless Cylindrical Shells," by MARIO G. SALVADORI and A. D. ATESHOGLOU, discusses cylindrical shell roofs. When the spans are relatively small or the shell is shotcreted, it is economical to extend the shell to the ground and eliminate the ribs. This paper evaluates maximum bending moments; thrusts; and vertical reactions due to dead load, live load, temperature variations, and wind in this type of structure.

R. E. PHILLEO presents a "Comparison of Results of Three Methods for Determining Young's Modulus of Elasticity of Concrete." The elastic response of concrete to static, resonance, and pulse velocity tests is discussed. Because of the wide variations in static testing techniques and the heterogeneity of concrete, the results of Young's modulus calculated from the three methods do not necessarily agree.

Title No. 51-15

A report of research sponsored by the Reinforced Concrete Research Council

Shear Strength of Reinforced Concrete Beams*

Part 1—Tests of Simple Beams

By K. G. MOODY,† I. M. VIEST,‡ R. C. ELSTNER,§ and E. HOGNESTAD§

SYNOPSIS

Data are presented on the shear strength of 42 simple beams, 40 without web reinforcement and two with web reinforcement. Tests were carried out in three series with the following variables: (1) percentage of longitudinal and web reinforcement, and method of anchorage; (2) size and percentage of longitudinal reinforcement and strength of concrete; and (3) concrete mixture and method of curing. The size of specimen was different for each of the three series.

The beams were tested with one or two concentrated loads and all failed in shear after one or more diagonal tension cracks formed in the region of maximum shear. The magnitude of the loading causing the initial diagonal tension cracks depended primarily on the cross section and the concrete strength. Most beams were able to sustain greater loads than the cracking loads. Magnitude of failure loads depended on cross section dimensions, amount of longitudinal reinforcement, amount of web reinforcement, strength of concrete, and length of shear span. Failure was by destruction of the compression zone of concrete above the diagonal tension crack and adjacent to a loading block.

Test results indicated that strength of beams with large *a-d* ratios may be governed by the load causing first cracking whereas the strength of shorter beams is governed by the load causing the destruction of the compression zone of concrete. Results also indicate that the load at first cracking may be predicted on the basis of nominal shearing stress and the ultimate load may be predicted on the basis of ultimate moment.

INTRODUCTION

The problem of the strength and behavior of reinforced concrete beams subject to combined shear and flexure has received considerable attention in the past.[1] Studies made by the Subcommittee on Ultimate Load Design of the ASCE Committee on Masonry and Reinforced Concrete indicated, however, that available experimental data on diagonal tension and shear were

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inconclusive and inadequate for predicting the ultimate strength of such beams. At the time these committee studies were made, the only equation available for predicting the ultimate strength of reinforced concrete beams failing in shear was that developed by Moretto[2] on the basis of his tests of simple beams.* It was considered desirable to find out whether Moretto's equation or some other equation of similar type is applicable also to restrained beams. With this in mind, a cooperative investigation was started in 1950 at the University of Illinois under the sponsorship of the Reinforced Concrete Research Council and the Bureau of Public Roads.

Preliminary studies of the results of these tests indicated that neither Moretto's equation nor any other equation of similar type was generally applicable; the results indicated also a need for coverage of a wider range of variables. Therefore, in 1951 the test program for the investigation of restrained beams was extended. Altogether, tests were made on 96 restrained beams and 14 simple beams. In addition, during 1951-52 tests were made on 28 simple beams as a part of the regular research work in the Department of Theoretical and Applied Mechanics of the University of Illinois. The tests of the latter investigation form a useful supplement to the tests made as a part of the cooperative project.

Results of the complete investigation are reported in four parts; the tests and their results are reported in the first three parts and analyzed in the fourth. Data from the tests of 40 simple beams without web reinforcement and 2 with web reinforcement are presented in Part 1. Part 2 includes data on tests of 61 restrained beams without web reinforcement; Part 3 includes results of the tests of 35 restrained beams with web reinforcement; and Part 4 includes a theoretical analysis of ultimate shear strength and comparisons of the analysis with test data obtained from this and several earlier investigations.[2,3,4] Part 4 also includes empirical equations for predicting loads at which diagonal tension cracks form.

SPECIMENS, EQUIPMENT, AND TEST PROCEDURES

Test program and specimens

The tests of 42 simple beams described herein were made in three series. (1) Series III, comprising 14 simple beams, was made as a part of the investigation of restrained beams failing in shear. (2) Series A, comprising 12 beams, was made to study the effects of the size of reinforcing bars on shear strength. (3) Series B, comprising 16 beams, was made to investigate the effects of different concrete mixtures and methods of curing on shear strength.

Series III.—All specimens of this series were rectangular beams 10 ft long and 7 x 24 in. in cross section (Fig. 1a). Three different percentages of tension reinforcement were included as shown in Table 1. The percentage of the compression reinforcement was equal to one-half of the tension reinforcement except in beam III-28b in which the compression reinforcement was omitted. Beams III-30 and III-31 were provided with vertical stirrups. The remaining beams were made without web reinforcement. The same concrete mix was used for all specimens.

This series included a study of the effects of anchoring the tension reinforcement with hooks. Half of the beams without web reinforcement were made with hooks, the other half with straight bars only as indicated in Fig. 1a and in Table 1.

*A similar equation was developed by A. P. Clark at a later date.[3]

Series A—All specimens of this series were rectangular beams 6 ft 3 in. long and 7 x 12 in. in cross section (Fig. 1b). The beams were reinforced with straight bars in tension only. Percentage of reinforcement and bar sizes were varied as shown in Table 2. Concrete strength was an additional variable.

Series B—All specimens of this series were rectangular beams 10 ft long and 6 x 12 in. in cross section (Fig. 1c). They were reinforced with two #7 bars in tension only. Type of cement, cement-water ratio, percentage of cement paste, and type of curing were the major variables as shown in Table 3.

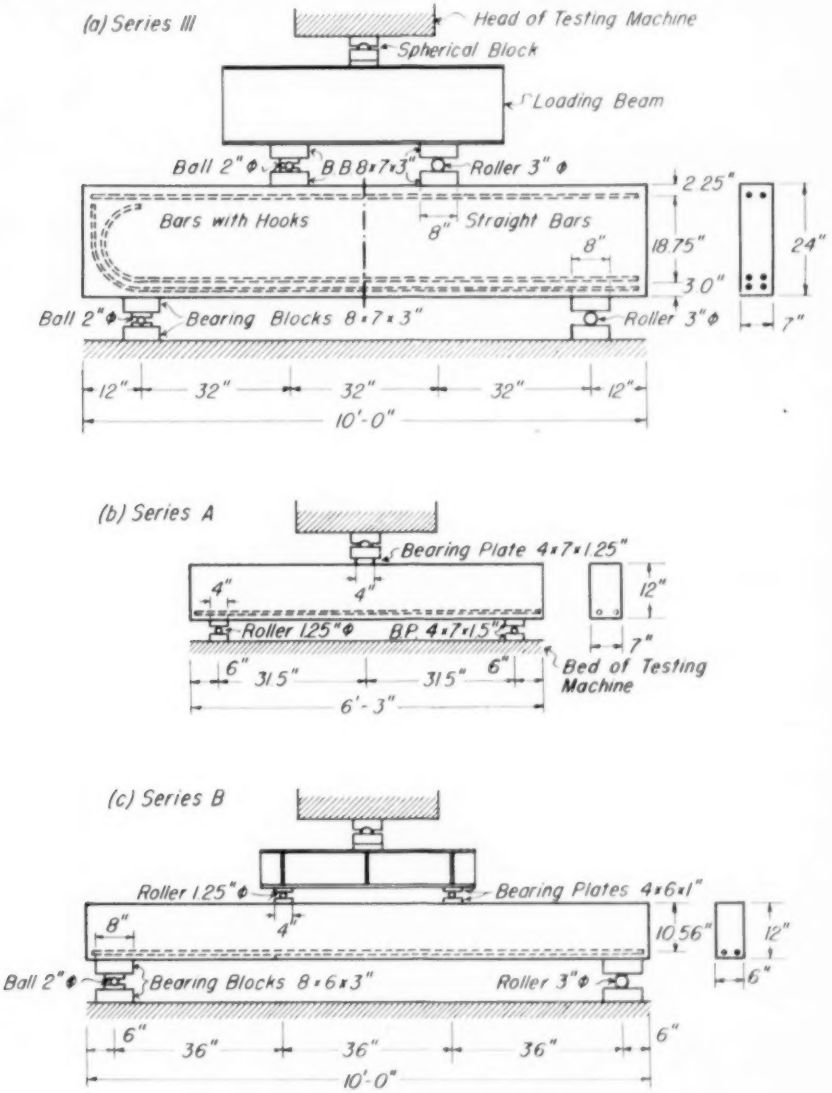

Fig. 1—Details of beams and testing arrangements

TABLE 1—DETAILS OF SPECIMENS—SERIES III*

Beam No.†	Longitudinal reinforcement		Vertical stirrups	
	Number and size of bars	p,‡ percent	Size and spacing	r_v percent
III—24a, b	4—# 9 H**	2.72	None	0
25a, b	4—#10 H	3.46	None	0
26a, b	4—#11 H	4.25	None	0
III—27a, b	4—# 9 S**	2.72	None	0
28a, b§	4—#10 S	3.46	None	0
29a, b	4—#11 S	4.25	None	0
III—30	4—#11 H	4.25	#3 at 6 in.	0.52
31	4—#11 H	4.25	#4 at 6 in.	0.95

*All beams were 7 x 24 in. in cross section and 10 ft long. Reinforcement was made of intermediate grade deformed bars. The effective depth was 21 in. and the percentage of compression reinforcement was equal to one-half of the tension reinforcement. For arrangement of reinforcement see Fig. 1a.
†In the designation of specimens letters a and b distinguish between two companion specimens.
‡Refers to tension reinforcement.
§No compression reinforcement in beam III—28b.
**H—hooked bars; S—straight bars.

TABLE 2—DETAILS OF SPECIMENS—SERIES A*

Beam No.	Effective depth d, in.	Longitudinal reinforcement	
		Number and size of bars	p, percent
A1	10.30	1—#11	2.17
A2	10.50	2—#8	2.15
A3	10.55	2—#7, 1—#6	2.22
A4	10.63	4—#6	2.37
B1	10.50	1—#8, 2—#4	1.62
B2	10.55	2—#7	1.63
B3	10.63	2—#6, 1—#5	1.60
B4	10.69	4—#5	1.66
C1	10.55	1—#7	0.81
C2	10.70	2—#5	0.83
C3	10.75	3—#4	0.80
C4	10.80	2—#4, 2—#3	0.82

*All beams were 7 x 12 in. in cross section and 6 ft 3 in. long. Reinforcement was intermediate grade deformed bars. For arrangement of reinforcement see Fig. 1b.

TABLE 3—DETAILS OF SPECIMENS—SERIES B*

Beam No.	Concrete mixture			Curing method
	Type of cement	C/W by weight	Percentage of paste†	
1	I	1.7	29	Dry
2	I	1.1	29	Dry
3	I	1.7	24	Dry
4	I	1.1	24	Dry
5	I	1.7	29	Wet
6	I	1.1	29	Wet
7	I	1.7	24	Wet
8	I	1.1	24	Wet
9	III	1.7	29	Dry
10	III	1.1	29	Dry
11	III	1.7	24	Dry
12	III	1.1	24	Dry
13	III	1.7	29	Wet
14	III	1.1	29	Wet
15	III	1.7	24	Wet
16	III	1.1	24	Wet

*All beams were 6 x 12 in. in cross section and 10 ft long. Reinforcement was two #7 intermediate grade deformed bars. Effective depth of reinforcement was 10.56 in. and percentage 1.89 percent. For arrangement of reinforcement see Fig. 1c.

†Percentage of paste $= \dfrac{\text{Volume of paste (Cement + Water)}}{\text{Total volume}}$.

TABLE 4—PROPERTIES OF CONCRETE MIXTURES

Series	Beams	Mix ratio by weight*	C/W by weight	Average slump, in.	Average f_c', psi†
III	All	1:3.67:5.21	1.21	3.2	3180
A	A	1:2.28:3.10	2.00	6.0	4490
	B	1:3.37:4.08	1.42	6.0	2860
	C	1:5.04:6.53	0.93	4.0	950
B	4, 8, 12, 16	1:4.41:6.10	1.10	3.8	2330
	2, 6, 10, 14	1:3.36:4.64	1.10	8.4	2860
	3, 7, 11, 15	1:3.17:4.39	1.70	0.5	4790
	1, 5, 9, 13	1:2.44:3.36	1.70	2.2	5310

*Cement: sand: gravel.
†For f_c' of individual specimens see Table 6.

Materials

Beams of Series III and A were made with Type I portland cement; beams of Series B were made with Type I and Type III portland cements as shown in Table 3. All specimens were made with Wabash River torpedo sand and Wabash River gravel. Specific gravities of the sand and gravel were 2.65 and 2.70, respectively, the fineness modulus of the sand was 3.34, and the maximum size of gravel was 1 in. Both aggregates passed the usual ASTM tests. Average properties of the concrete mixtures are given in Table 4. Concrete strengths for the individual beams are presented with the beam test results in Table 6.

Deformed bars of intermediate grade steel meeting ASTM Designation A305-50T were used for all longitudinal and web reinforcement. Control tests of tensile coupons were made only for Series III, and the results are given in Table 5. No tests were made of the steel for beams of Series A and B; such tests were not needed since the reinforcement did not yield in any corresponding beam test.

Fabrication of specimens

All beams were cast in steel forms. The bottom bars were supported from the base of the forms on steel chairs. The top bars for Series III were supported rigidly from the side forms. For beams with web reinforcement, the reinforcement was tied into a rigid cage before it was placed in the forms.

The concrete was mixed in a nontilting horizontal drum mixer of 6½-cu ft capacity. All batching was done by weight. The concrete was placed in the forms with the aid of internal vibrators. Forms were removed one day after casting. The beams of Series III and A were then subjected to wet curing for 6 days, followed by storage in the air of the laboratory until testing took place at the age of approximately 28 days. The beams of Series B were subjected either to "dry" or to "wet" curing. The dry cured beams were placed under wet burlap after the forms were removed; the burlap was covered with building paper to prevent rapid evaporation of moisture. The covers were removed 7 days after casting and the beams were stored in the air of the laboratory until tested at the age of 28 days. The wet cured beams were kept in a saturated condition under wet burlap until tested at the age of 28 days.

Three 6 x 12-in. control cylinders were made from each batch of concrete. The cylinders were cast in steel forms and the concrete was compacted with the aid of internal vibrators. The cylinders were cured in an identical manner and tested on the same day as the corresponding beams.

TABLE 5—PHYSICAL PROPERTIES OF REINFORCEMENT—SERIES III*

Bar size	Number of tests	Average yield point, psi	Average ultimate strength, psi	Average elongation in 8 in., percent
3	52	47,300	72,500	21.0
4	51	44,000	74,000	19.8
9	8	45,700	74,200	26.0
10	9	45,400	77,700	24.1
11	7	43,800	76,000	23.8

*All reinforcement was intermediate grade deformed bars.

Test equipment and procedures

All beams were tested in a 300,000-lb Olsen machine of the beam type. In Series III, the load was applied at the third-points of an 8-ft span, in Series A at the center of a 5-ft 3-in. span, and in Series B at the third-points of a 9-ft span. Details of the test apparatus are shown in Fig. 1.

In Series III, the load was usually applied in 5000- or 10,000-lb increments until failure. After each increment the machine was operated intermittently to maintain the load constant while cracking and creep of the concrete proceeded. After the load was stabilized, strains in the tension and web reinforcement, strains on the top surface of the beam, and deflections were measured, and crack patterns were noted. Deflections were measured at midspan and at the load points with the aid of a frame mounted on the beam over the supports and equipped with 0.001-in. dial indicators. Strains were measured with SR-4 electric resistance strain gages connected to a Baldwin SR-4 strain indicator through a switch box. Type A-11 1-in. gages were mounted on the tension steel near midspan and on the web reinforcement after the concrete had hardened; for this purpose core holes were provided in the concrete during casting. Type A-9 6-in. gages were attached to the top surface of the beam near midspan a few days before testing. Total testing time varied from ½ to 4 hr.

In Series A, the load was applied in 1000-lb increments until failure. Deflections at midspan were measured after each increment with a 0.001-in. dial gage bearing against the bed of the testing machine. The progress of cracking and the phenomenon of failure were observed visually.

In Series B, the load was applied in 1000-lb increments until failure. Strains in the tensile reinforcement at midspan were measured in a similar way as in beams of Series III. The progress of cracking and the phenomenon of failure were observed visually.

TEST RESULTS

Results of the tests are summarized in Table 6 which includes the average concrete strength f_c', the loads at the formation of the initial diagonal tension crack P_c, the failure loads P_u, and the critical steel stresses at failure f_s. In addition, the table includes the nominal shearing stress v_c and the ratios of the nominal shearing stresses v_c, and v_u to the concrete strength f_c'.

Concrete strengths given in Table 6 are the average values determined from the tests of all control cylinders made with any particular beam. Cracking loads P_c were determined from the load-deflection curves for beams without stirrups and from load-stirrup strain curves for beams with stirrups.* For beams of Series B, such curves were not available, and the loads P_c recorded in Table 6 are based on visual observations of cracking. The failure loads P_u were measured by the testing machine. The critical steel stresses were obtained from steel strains measured at midspan.† They represent average quantities which, if multiplied by the total area of the tension reinforcement, give the tensile force in the steel at sections at which failure occurred. Since the determination of f_s involved strains measured at sections other than the critical, f_s is an approximate value which should be used only as a guide. Nominal shearing stresses v_c and v_u were computed from the equation $v = 8V/7bd$, in which V is the maximum shear corresponding to

*Load-deflection and load-strain curves exhibited a definite break at P_c.

†The critical stresses were determined as follows. First, the maximum stresses measured at midspan were averaged and extrapolated to failure load. Since the failure section is not exactly the section of maximum moment, the average strains for midspan were then reduced by the ratio of the moment at the failure section to the moment at midspan. These critical strains were converted into stresses with the aid of modulus of elasticity determined from tests of coupons.

the appropriate value of load P ($V = \frac{1}{2}P$), and b and d are the width and the effective depth of the beam.

Load-deflection and load-stirrup strain curves for a few specimens are presented in Fig. 2 and 3; a few representative specimens after failure are shown in Fig. 4 and 5.

Behavior prior to formation of diagonal cracks

The behavior of all beams was essentially elastic until tension cracks formed.

TABLE 6—TEST RESULTS

Beam No.	Concrete strength f'_c, psi	Initial diagonal tension cracking			Failure			P_u/P_c
		P_c, kips	v_c, psi	v_c/f'_c	P_u, kips	Critical steel stress f_s, ksi*	v_u/f'_c	
Series III								
III—24a	2580	40.0	156	0.060	133	24.8	0.200	3.33
b	2990	50.0	195	0.065	136	26.2	0.175	2.72
III—25a	3530	55.0	214	0.061	120	18.4	0.133	2.18
b	2500	45.0	175	0.070	130	20.0	0.202	2.89
III—26a	3140	60.0	234	0.075	189	26.0	0.236	3.15
b	2990	50.0	195	0.065	178	23.0	0.231	2.56
III—27a	3100	45.0	175	0.057	156	28.5	0.198	3.47
b	3320	50.0	195	0.059	160	29.5	0.189	3.20
III—28a	3380	50.0	195	0.058	136	21.0	0.158	2.72
b†	3250	45.0	175	0.054	153	26.1	0.184	3.40
III—29a	3150	60.0	234	0.074	175	22.4	0.216	2.92
b	3620	60.0	234	0.065	196	24.1	0.213	3.27
III—30	3680	50.0	195	0.053	215	——	0.220	4.30
III—31	3250	50.0	195	0.060	228	——	0.274	4.56
Series A								
A1	4400	26.0	206	0.047	27.0	——	0.049	1.04
2	4500	30.0	233	0.052	30.0	——	0.052	1.00
3	4500	28.0	217	0.048	34.0	——	0.058	1.21
4	4570	30.0	231	0.051	32.0	——	0.054	1.06
B1	3070	22.9	178	0.058	25.3	——	0.064	1.10
2	3130	27.0	209	0.067	27.0	——	0.067	1.00
3	2790	24.0	184	0.062	25.0	——	0.065	1.04
4	2430	23.8	182	0.075	25.0	——	0.079	1.05
C1	920	9.0	70	0.076	9.0	——	0.076	1.00
2	880	11.0	84	0.095	11.0	——	0.095	1.00
3	1000	11.4	87	0.087	11.4	——	0.087	1.00
4	980	11.3	85	0.087	11.3	——	0.087	1.00
Series B								
B-1	5320	23.0	208	0.039	26.0	40.9	0.044	1.13
2	2420	15.0	136	0.056	16.0	24.3	0.060	1.07
3	3740	23.0	208	0.056	23.5	36.9	0.057	1.02
4	2230	17.0	154	0.069	18.2	29.8	0.074	1.07
5	4450	21.0	190	0.043	23.4	33.3	0.047	1.11
6	2290	15.0	136	0.059	15.5	21.6	0.061	1.03
7	4480	20.0	181	0.040	23.0	31.8	0.046	1.15
8	1770	14.0	127	0.072	14.0	19.3	0.072	1.00
B-9	5970	23.0	208	0.035	24.0	36.1	0.036	1.04
10	3470	19.0	172	0.050	22.0	37.3	0.057	1.16
11	5530	22.0	199	0.036	27.0	43.4	0.044	1.23
12	2930	21.0	190	0.065	21.2	31.6	0.066	1.01
13	5480	20.0	181	0.033	25.0	37.9	0.041	1.25
14	3270	19.4	175	0.054	19.4	31.7	0.054	1.00
15	5420	23.0	208	0.038	23.0	33.6	0.038	1.00
16	2370	16.0	145	0.061	17.0	29.5	0.065	1.06

*Estimated from strains measured at or near midspan.
†No compression reinforcement.

Tension cracks were initiated on the bottom surface of the beams and spread vertically upward. As the load increased, tension cracks located between the supports and the nearest load points curved toward the load point (Fig. 5). During this initial development of tension cracks, the beams behaved as would be expected for any reinforced concrete flexural member cracked in

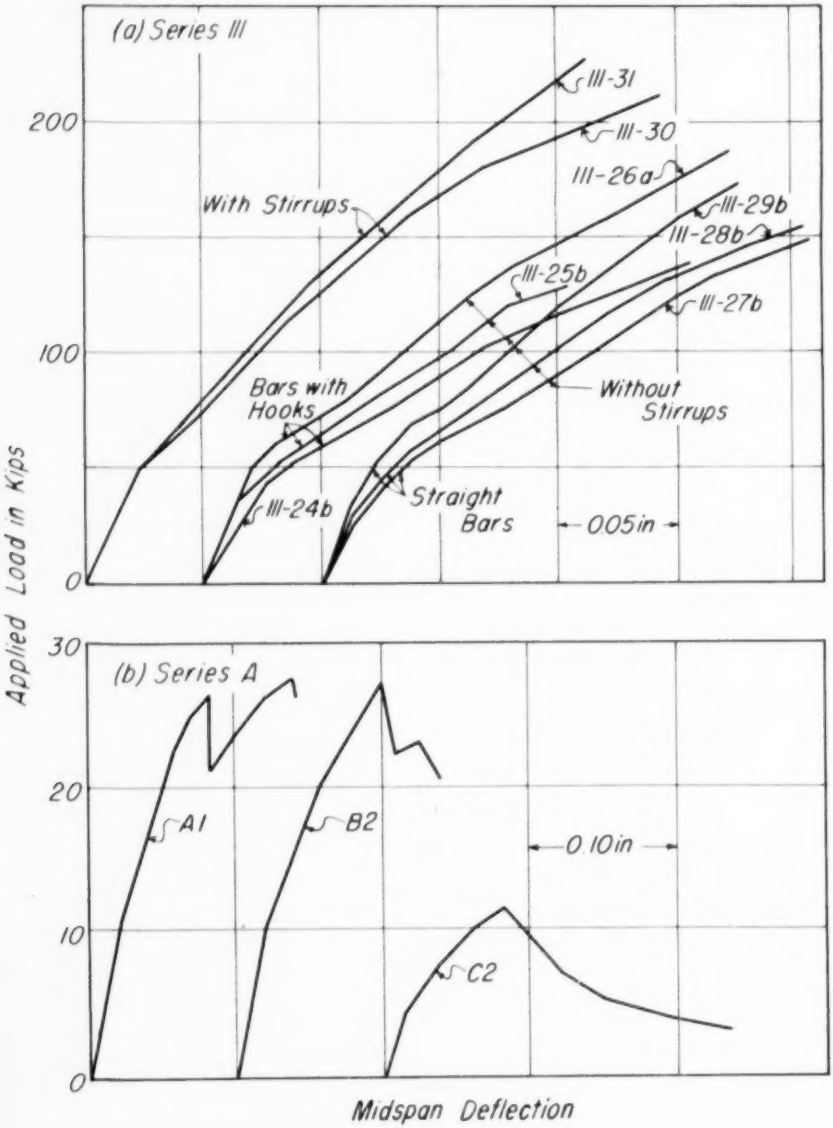

Fig. 2—Load-deflection curves

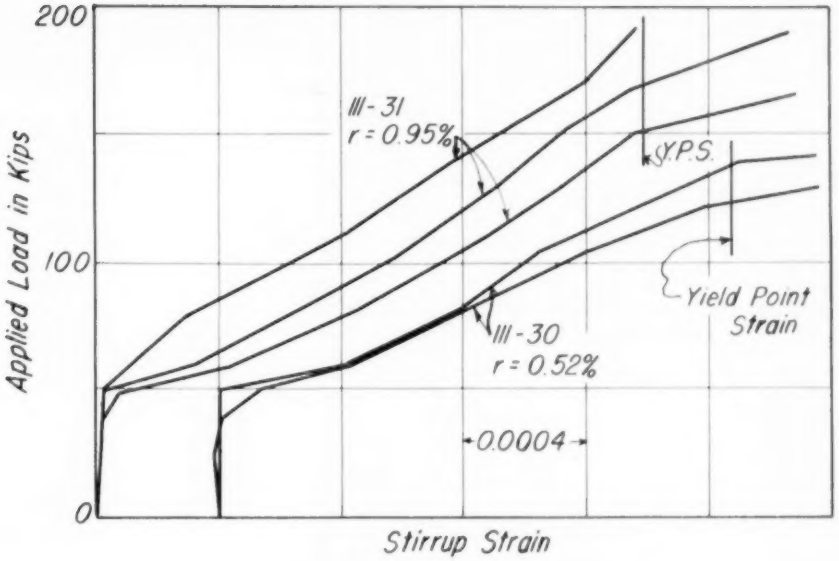

Fig. 3—Load-stirrup strain curves

Beam III-25a

Beam III-27b

Beam III-28b (no compression reinforcement)

Beam III-31 (with web reinforcement)

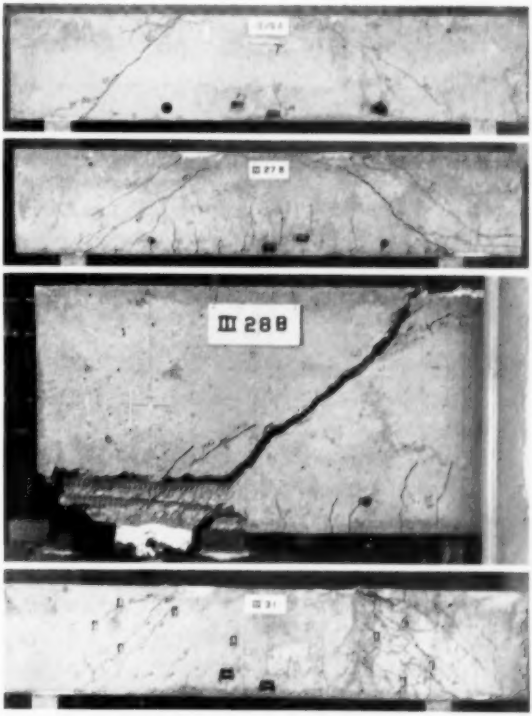

Fig. 4—Beams of Series III after failure

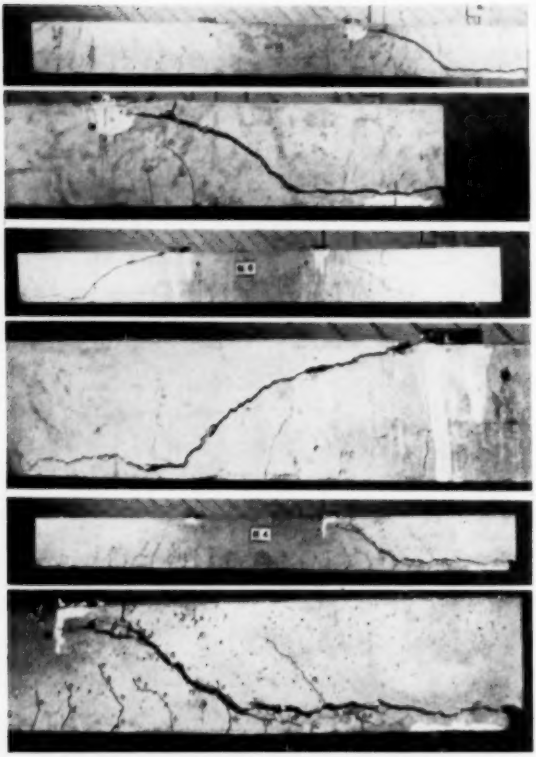

Fig. 5—Beams of Series B after failure

tension. This stage of behavior was ended when inclined cracks, hereafter called initial diagonal tension cracks, formed.

The presence of web reinforcement had no important effect on the behavior of beams prior to the formation of initial diagonal tension cracks. This is illustrated in Fig. 2 and 3 which show that throughout this range of loading (1) the load-deflection curves were parallel for comparable beams with and without stirrups and (2) the strains in the stirrups were practically equal to zero.

Initial diagonal tension cracking

For the purposes of this paper, a diagonal tension crack is defined as a major inclined crack; this crack causes a significant redistribution of internal stresses.* Several diagonal tension cracks may form in a beam before the ultimate load is reached. The one which forms first is called the initial diagonal tension crack and the corresponding load is called the initial diagonal tension cracking load.

Diagonal tension cracks observed in these tests had certain common characteristics by which they could be distinguished from ordinary tension cracks.

*The phenomenon of redistribution of internal stresses is discussed in the latter part of this paper.

In all beams the diagonal tension cracks were initiated at or above the tension reinforcement and were inclined from their inception. As the load increased, the diagonal tension cracks developed further at both ends. At one end they extended toward the load point, usually passing over the ends of the tension cracks. At the other end (located near the tension steel) they either crossed the reinforcement at an angle of between 30 and 60 deg as illustrated in Fig. 4 or extended along the tension reinforcement causing horizontal splitting illustrated in Fig. 5. In a few beams the diagonal tension cracks originated at some point on an existing tension crack, but even in these cases they assumed the characteristics described above.

In long beams, the diagonal tension cracks were initiated midway between the supports and load points. In several beams of Series III, the starting point for the diagonal tension crack was nearer to the support.

The initial diagonal tension cracking loads (hereafter called cracking loads) are listed in Table 6. It can be seen from the results of Series III that the cracking load is independent of the presence of hooks on the longitudinal reinforcement and of the presence of vertical stirrups. A slightly increasing trend with the percentage of web reinforcement may be observed in Series III, but the data are erratic and inconclusive. The results of Series A and B show that the strength of concrete has the major influence on the cracking load; the cracking load increases with increasing strength of concrete. The results of Series A indicate also that the cracking load may be higher when a given area of tension reinforcement is furnished with a larger number of small bars.

A comparison of the ratios v_c/f_c' for the three series of beams indicates a small increase of these ratios when the ratio of the shear span to the effective depth of beam* decreases. Furthermore, it may be noted that the ratio v_c/f_c' varied from 0.033 to 0.095 with the majority of values smaller than 0.060. The minimum value for beams with concrete strengths smaller than 3000 psi was 0.056, with f_c' smaller than 4000 psi it was 0.050, and with f_c' smaller than 5000 psi it was 0.40. The present allowable value of nominal shearing stress is $0.030f_c'$.[5]

Behavior after formation of diagonal cracks

The ratio of the ultimate load to the cracking load is shown in column P_u/P_c of Table 6. For six beams of Series A and for three beams of Series B the cracking and the ultimate loads were equal. For the remaining beams of these series and for all beams of Series III the ultimate load was higher than the cracking load. The behavior of beams after the formation of diagonal tension cracks is described separately for each series of beams.

Series III—In beams without web reinforcement the formation of the initial diagonal tension crack was followed by formation of a similar crack on the opposite half of the beam. With further increases of load, both cracks extended upward toward the load point, and at loads approaching the ultimate

*The shear span is the distance between a reaction and the nearest load.

the upper ends of the diagonal tension cracks reached to within a few inches of the upper beam surface (Fig. 4, beam III-25a). In several beams, a second set of diagonal cracks formed above the original cracks either before or simultaneously with failure (Fig. 4, beam III-27b). The failure occurred always by destruction of the concrete located above the upper end of one diagonal crack. It invariably took place alongside the loading block, in most beams along the edge closer to the adjacent support.

One of the beams without stirrups, III-28b (Fig. 4), had no compression steel. The failure of this beam was very sudden and was accompanied by splitting of the concrete along the tension reinforcement.

The behavior of beams with web reinforcement differed from that of beams without web reinforcement in that several diagonal tension cracks formed instead of one or two (Fig. 4, beam III-31). These cracks were closely spaced and approximately parallel to the direction of the initial diagonal cracks.

At failure, the upper ends of the diagonal cracks were closer to the top surface of the beam than the upper ends of the tension cracks, indicating that the beams failed at loads lower than those corresponding to their flexural capacities. This finding was confirmed by strains measured at midspan on the top surface of the beam and on the tension steel. Concrete strains never exceeded 0.0006 and steel strains were always smaller than 0.0013 at these locations, whereas the corresponding values for flexural failures should be approximately 0.0040 and 0.0016, respectively.

Typical load-deflection curves are shown in Fig. 2a. Typical load-stirrup strain curves are shown in Fig. 3.

Series A—In six beams of this series the cracking load was the highest the beams could carry; in the remaining six beams some increases of load were possible after diagonal tension cracking occurred. In all beams the initial diagonal cracks formed suddenly, often assuming immediately their final shape. At failure, the concrete located adjacent to the central loading block and above the upper end of the diagonal crack was destroyed and splitting occurred along the tension reinforcement.

Load-deflection curves for three beams are shown in Fig. 2b. The curve for beam A-1 is typical of all beams with the ultimate load higher than the cracking load. The curve for beam C-2 is typical of all beams with the ultimate load equal to the cracking load except for beam B-2; the curve for beam B-2 is also shown.

Series B—The behavior of beams of this series was similar to that described for beams of Series A. The shape of the diagonal tension cracks and the manner of failure are illustrated in Fig. 5. Beam B-14 in this figure is representative of beams with the ultimate load equal to the cracking load, and beams B-6 and B-4 are representative of beams in which the ultimate load was higher than the cracking load. In beam B-6 the destruction of concrete occurred along the loading block edge closer to the adjacent support, whereas in beam B-4 the diagonal crack passed under the loading block and the concrete was destroyed at the edge further away from the adjacent support.

Redistribution of internal stresses

Until diagonal tension cracks form, the stresses in the tension steel and in the concrete are distributed along the length of the beam in the same way as the external moments so that these stresses at any section are approximately proportional to the moment at that particular section. The formation of diagonal tension cracks changes these relationships. Such changes are called the redistribution of internal stresses.*

In Fig. 6a a beam without web reinforcement is shown after formation of diagonal tension cracks. The part of the beam located to the left of one crack is shown in Fig. 6b as a free body. Since no stresses can exist across the crack, the free body is subject to the action of the reaction of the support R, the force in the tension reinforcement T, the compressive force C' resisted by the concrete above the crack, and the vertical shear V. It is indicated in Fig. 6b that all vertical shear is transmitted through the concrete; in reality a part of the shear is transmitted through the dowel action of the reinforcement, but the contribution of the tension reinforcement to the transfer of shear is believed to be negligible.

If the forces shown in Fig. 6b are in equilibrium, then

$$A_s f_s jd = Ra;$$

that is, after the formation of the diagonal crack the steel stress at section b-b depends on the moment at section a-a. Consequently, the steel stress at section b-b increases, which increase is accompanied by widening of the diagonal crack. It is believed that the widening of the diagonal crack leads to a localized increase of the compressive stress in the concrete at section a-a. Thus in a beam with diagonal tension cracks the distribution of stresses in the tension reinforcement and in the concrete along the beam does not follow the distribution of external moments.

When a diagonal tension crack forms, the redistribution of internal stresses discussed above takes place. Such redistribution may lead either to an immediate failure or to a stable condition permitting further increases of load.

Failure

A reinforced concrete beam failing as a result of

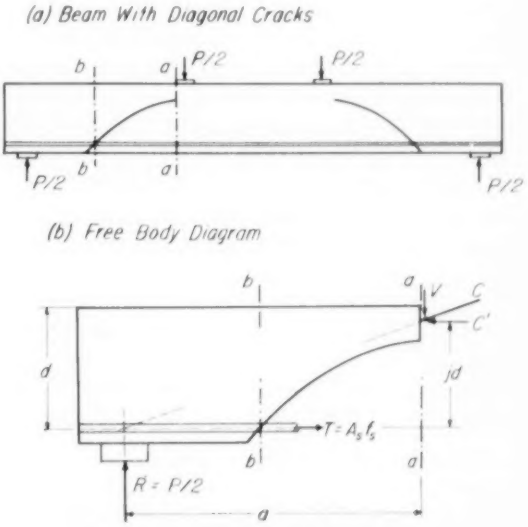

Fig. 6—Redistribution of internal stresses

*Redistribution of internal stresses caused by diagonal tension cracks was discussed first by Mörsch in 1907.[8]

the presence of shearing forces may fail by one of the following modes: (1) diagonal tension failure; (2) destruction of the compression zone; (3) bond failure; or (4) splitting failure.

Diagonal tension failures—These failures have been observed frequently in laboratory tests. Beams are said to fail in this manner when the initial diagonal tension cracks form at the maximum load sustained by the beam.

In beams of Series III, that is in beams with small *a-d* ratios, the ratio of the maximum load to the load of first cracking, P_u/P_c, was always higher than 2. In beams of Series A and B with larger *a-d* ratios, the ratio of P_u/P_c was either equal to 1 or slightly higher. Thus, it appears that beams with large *a-d* ratios may fail at the cracking load.

Destruction of the compression zone—In a beam failing by this type of failure diagonal tension cracks form, redistribution of internal stresses takes place, and a new equilibrium of internal and external forces is reached. Further increases of load are possible until the concrete located above the diagonal crack is destroyed under the action of force C (Fig. 6b). Since this concrete is stressed in compression, the failure is said to occur by the destruction of the compression zone of concrete.

In the shear tests reported in the literature, this mode of failure was predominant. The majority of beams tested in this investigation failed also in the same manner; the corresponding loads for beams of all three series are given in Table 6.

It can be seen from the results of Series III that the failure load P_u increased with increasing percentage of longitudinal reinforcement and with increasing percentage of web reinforcement. The results of Series B and A showed that the load P_u increased also with the increasing strength of concrete. The magnitude of the ultimate load seemed to be more sensitive to the changes in the strength of the concrete than to the changes in the percentage of tension reinforcement.

The ratios v_u/f_c', also listed in Table 6, show that the nominal shearing stress was considerably higher at failure of beams of Series III than at failure of beams of either Series A or B. For beams of Series B, v_u/f_c' varied from 0.036 to 0.074, for beams of Series A from 0.049 to 0.087, and for Series III from 0.133 to 0.236 for beams without web reinforcement and 0.229 to 0.274 for beams with web reinforcement. The corresponding allowable value of the nominal shearing stress for beams without web reinforcement is $0.030\ f_c'$.[5]

The ratios of the effective beam depth to the shear span (d/a) for Series B, A, and III were 0.29, 0.33, and 0.66, respectively. A comparison between these values and the respective v_u-f_c' ratios suggests a direct proportionality. If A is the constant of proportionality, then

$$\frac{v_u}{f_c'} = A\frac{d}{a}$$

After substituting for the nominal shearing stress and rearranging, this equation may be written as

$$V_u a = B\,bd^2 f_c'$$

where B is a numerical constant. Since $V_u a$ is the ultimate moment M_u at the section of failure, the right hand side of the last equation suggests that for shear failures by the destruction of the compression zone the resisting moment M_u may be independent of the magnitude of shear.

Bond failures—Bond failures were observed by Talbot[7] in his early investigations. Reference to Fig. 6a and to the discussion of the redistribution of internal forces shows that the formation of the diagonal tension crack is accompanied by a considerable increase of the tension force T in the reinforcement at section b-b. For relatively short beams reinforced with bars of poor bond characteristics this may lead to secondary bond failures. An anchorage of the reinforcement, such as hooks, has previously been found to reduce the danger of this type of failure.

None of the beams reported herein appear to have failed in anchorage bond even though the bond stresses magnified by the formation of the diagonal crack may have been of the order of 600 psi or more. An indication of the absence of bond failures may be obtained by comparing beams of Series III with and without hooks on the tension reinforcement:

Bars with hooks			Bars without hooks		
Beam	Concrete strength, psi	Ultimate load, kips	Beam	Concrete strength, psi	Ultimate load, kips
24a	2580	133	27a	3100	156
b	2990	136	b	3320	160
25a	3530	120	28a	3380	136
b	2500	130	b	3250	153
26a	3140	189	29a	3150	175
b	2990	178	b	3620	196
Average	2960	148	Average	3300	163

It can be seen clearly that the ultimate loads for the two groups of beams are nearly equal when the differences in the strength of concrete are taken into account. Thus, these tests indicate that with modern deformed bars bond failures are not probable in simple beams of normal proportions.

Splitting failures—This type of failure is caused by horizontal splitting of the beam along the tension steel. The major factors believed to contribute to splitting failures are the dowel action of the reinforcement and the diagonal tension stresses caused by the transfer of stress from the reinforcement to the concrete by bond. Splitting along the horizontal bars was observed in several beams reported herein either at or after failure. However, it is believed that splitting was a secondary rather than a primary phenomenon in these tests.

CONCLUDING REMARKS

Forty-two simple beams were tested with one or two concentrated loads and all failed in shear after one or more diagonal tension crack formed in the region of maximum shear. Up to the formation of diagonal tension cracks the behavior of all beams was the same as that of beams failing in flexure. The formation of diagonal tension cracks led to a new distribution of internal stresses which prevailed until failure.

The magnitude of the load causing the formation of initial diagonal tension cracks depended primarily on the dimensions of the cross section and on the strength of the concrete. Most beams were able to sustain higher loads than this cracking load. At failure of all beams the compression zone of concrete was destroyed above the diagonal tension crack and adjacent to one of the loading blocks. The magnitude of the failure loads depended primarily on the dimensions of the cross section, on the amount of longitudinal reinforcement, on the amount of web reinforcement, on the strength of concrete, and on the length of shear span.

The ratio of the ultimate load to the load at first cracking decreased as the ratio of the shear span to the effective depth increased. Test results indicated that the strength of beams with large a-d ratios may be governed by the load causing first cracking whereas strength of shorter beams is governed by the load causing destruction of the compression zone of concrete.

The results of these tests indicated further that the load at first cracking may be predicted on the basis of the nominal shearing stress and the ultimate load may be predicted on the basis of the ultimate moment. Analytical expressions serving this purpose are presented in Part 4 of this series.

ACKNOWLEDGMENTS

The tests reported in this paper were carried out at the University of Illinois. Those of Series III were made as a part of a cooperative investigation conducted by the University of Illinois Engineering Experiment Station under the sponsorship of the Reinforced Concrete Research Council of the Engineering Foundation and the Bureau of Public Roads of the U. S. Department of Commerce. The tests of Series A and B were made as a part of the regular research work of the Department of Theoretical and Applied Mechanics. This paper is based on the doctoral dissertation of the senior author.[8]

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Title No. 51-16

Effect of Algae Infested Water on the Strength of Concrete*

By BENJAMIN C. DOELL†

SYNOPSIS

The purpose of this study was to establish a relationship between loss in strength of concrete and various concentrations of organic matter contained in algae infested water. Various amounts were added to the mixing water of test batches with the resulting cylinders tested at 28 and 60 days. A close approximation of the allowable concentration of organic matter in a good mixing water was established.

INTRODUCTION

In the past, engineers on concrete jobs have frequently been confronted with the problem of permitting or condemning the use of certain types of contaminated water. In certain instances the contamination consisted of harmful salts and in other instances of various types of organic matter. Many times, when contamination consisted of organic matter, certain water was rejected because of either odor or appearance, causing needless work and expense in securing better water. This problem has become quite acute especially in the use of water from sloughs, dams, and dugouts in western Canada where the organic matter is in the form of algae. In studying the problem, the algae infested water of Duncairn Dam was chosen. It was here that the problem was most acute in the summer of 1952, the water containing a higher concentration of algae than usual.

The purpose of this report is to establish the over-all effect of the organic matter on the strength of concrete and not to concentrate on isolated phases which contribute to the loss in strength. As a result of the tests performed it is hoped to predict, with a larger measure of confidence, results of the use of such organic waters.

PREVIOUS INVESTIGATIONS

Mills[1] states that if the silt (in sand) contains more than 10 percent organic matter, the latter constituting as much as 1/10 of 1 percent of the sand, an appreciable injury results. Clay, if finely divided and uniformly distributed throughout the sand, appears to have little effect unless present in amounts greater than 10 percent. Mica is injurious even in small quantities.

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Williams[2] proposes that organic impurities (in sand) such as vegetable loam will always seriously diminish the strength of concrete. Even as little as 0.5 percent organic matter has been found to reduce the strength of mortar as much as 20 percent. Vegetable matter seems to coat sand grains and thus diminish the adhesion. Therefore, while clay may be ordinarily admitted up to 7 percent without objectionable results, all organic matter should be rigidly avoided.

Probably the most extensive research was reported by Abrams.[3] Strength tests of portland cement concrete were made at ages of 3 days to 2⅓ years using mixing waters of a wide range of types, many of which were thought to be unsuitable for use in concrete. Pertinent conclusions were:

". . . The quality of a mixing water is best measured by the ratio of its 28-day concrete or mortar strength to that of similar mixes with fresh water. While the lowest permissible strength-ratio is a matter of judgment, waters giving strength-ratios which in general fall below 85 percent should be considered unsatisfactory; if only isolated tests are made, 80 percent should be the limiting value. The time-of-setting test appears to be an unsafe guide as to the suitability of a water for mixing concrete.

". . . Sulphate waters produced little or no ill effects until a SO_4 concentration of about 1 percent was reached. For a concentration of 0.5 percent the average reduction in strength was about 4 percent; a concentration of 1 percent was required to produce a reduction in strength of more than 10 percent."

The 1940 report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete[4] states that when subjected to the mortar strength test, the strength at 28 days of mortar specimens made with the water under examination and normal portland cement shall be at least 90 percent of the strength of similar specimens made with the same cement and with water of known satisfactory quality. The *Handbook for Concrete and Cement*[5] suggests that acceptable mixing water should have a pH range of 6.0 to 8.0 and be free from organic matter.

Lyse[6] stated that organic impurities in concrete have generally been considered extremely detrimental, and a number of concrete structures have failed due to the presence of such impurities in the aggregate. He described tests in which it had been attempted to neutralize organic impurities by the addition of various chlorides. The results were interpreted as an indication that calcium chloride in particular was very suitable. Its use was recommended in amounts up to 2 percent of the cement by weight in construction of structures not requiring high strength concrete.

TYPES OF ALGAE

Algae comprise a large group of flowerless plants which form the main and characteristic vegetation in water. Some types live in sea water and some in fresh water, whether stagnant or running. They vary in size from the small microscopic plant, noticeable only because of the color it gives to the water, to masses with stemlike structures longer than the tallest trees. The two most common types encountered in western Canada are the globular or circular, unicellular algae and the branched, multicellular algae. The latter

possess leaflike structures which are formed of a delicate, perforated network much like lace or the skeletons of leaves. In the unicellular plant, reproduction (by cell division) and food manufacture take place within the cell, while in the multicellular algae a certain portion of its cells are devoted to specific duties. Each cell absorbs its own minerals and gases from the water and makes its food by photosynthesis. The cells of the algae are covered with a gelatinous, slimy material. The color of the algae is due to green pigments (chlorophyll) and red or blue pigments secreted in the cell. The diatom group of algae contains large quantities of silica. It is known that while algae live they give off oxygen and when they die carbon dioxide is formed as one of the products of their decomposition.

From this it seems there would be three possible ways in which algae could cause a decrease in the strength of concrete: (1) The organic matter could reduce the adhesion of the cement to the sand and aggregate. (2) The secretions or body fluids of the algae could react with certain chemicals in the cement to weaken its bonding qualities. (3) The surface active nature (or sudsing nature) of the organic compounds (including gelatinous covering) and algae could cause a high degree of entrapped air.

BASIC CONSIDERATIONS IN APPROACHING THE PROBLEM

For purposes of testing, a water-cement ratio of 0.5 (an average value) was selected. It was also decided to try to obtain concrete with a slump of 3 in. $\pm$ $\frac{1}{2}$ in., this being the average slump used on most jobs. This required a considerable number of trial batches, many of which resulted in $1\frac{1}{2}$-in. slumps. One reason for difficulty in obtaining the 3-in. slump was the small quantity of water required for the 150-lb batches. The other reason was that the procedure was accurately timed according to a certain set rate and if the mix was found to be too stiff a predetermined, preweighed quantity of water was added. Many times the first mixes became too wet because of the difficulty in predetermining the water required and its effect; these mixes were wasted. For this reason, the $1\frac{1}{2}$-in. slump mixes, when obtained, were kept. This conserved valuable materials and the cylinders were still of practical value since much concrete is placed at this slump. Therefore $1\frac{1}{2}$-in. slump mixes as well as 3-in. slump mixes were kept for testing and two groups of results obtained.

The next step was to select a range of organic concentrations. Because higher concentrations are seldom encountered, it was decided to use a range of 0 to 10 percent organic concentrate, considered to be the range in which there would be doubt as to the suitability of the water for mixing. Only concentrations of 0, 1, 3, 5, and $7\frac{1}{2}$ percent organic solution were used because of the lack of time and materials.

The only other controllable variable, the water content, had to be so adjusted that the resulting batch conformed to the basis decided on. This was done almost purely by trial and error although the 3 percent reduction in water content for each 1-in. excess slump was followed, as suggested in many

books on proportioning. In these tests this did not prove too helpful, as explained later.

Cylinder strengths were measured at intervals of 28, 60, and 90 days, and 1 year. To conform with standard testing practice, five cylinders were chosen as a minimum number for the 28- and 60-day groups. For the 90-day and 1-year tests, only two and three cylinders, respectively, were kept for testing since there was not enough room for more in the curing tanks. The latter specimens will be used for study by the Prairie Farm Rehabilitation Administration (PFRA) and correlation by the writer, with further testing proposed. The above schedule required 15 cylinders per batch.

Since 3 x 6-in. cylinders permitted the use of a sufficiently large aggregate,[7] conserved limited space, and test results could readily be converted into the usual 6 x 12-in. cylinder test results,[8] they proved most suitable and practical for purposes of these tests.

All cylinders were cured in city tap water at room temperature (68 F).

TEST MATERIALS AND PROCEDURES

Materials

To be on the safe side and still use an aggregate close to that used in actual field conditions,[7] a ¾-in. maximum size aggregate was selected. The aggregate was local pit-run gravel which, to meet ASTM specifications for coarse aggregates,[9] was screened into two sizes which were later blended on a 50-50 basis; that is, 50 percent of the aggregate passing a ⅜-in. sieve and retained by a No. 4 sieve was blended with 50 percent of that passing a ¾-in. sieve and retained by a ⅜-in. sieve. The aggregate consisted chiefly of sharp, angular stones and had a specific gravity of 2.67, water absorption of 1 percent, and a water content of 0.09 percent.

To the local sand was added 30 percent blending sand to meet ASTM grading requirements.[9] The fineness modulus of the sand was 2.68.

Cement was standard portland cement with a specific gravity of 3.15.

The mixing water was city of Saskatoon tap water mixed with the percentages (by weight) of organic concentrate required by the various tests. That is, 5 percent algae or organic solution referred to later on means that 5 percent of the weight of mixing water consisted of this organic concentrate.

Procuring of organic matter

The algae were secured from Duncairn Dam in southwest Saskatchewan at a time when the wind caused a particularly high concentration near the dam. Here the organic infested water was strained through burlap bags to obtain as concentrated a solution as possible. The separation of organic matter from the water was almost impossible since the slimy, finely divided and colloidal constituents tended to pass through this sieve. However, after several repetitions of sieving a high concentration of this material was obtained. This "algae-concentrate" was sealed in pails and immediately transported to the laboratory where it was thoroughly mixed, resealed, and placed in a cool place.

TABLE 1—ORGANIC (ALGAE) CONCENTRATE ANALYSIS*

Constituents	Amount, ppm	Percent by weight
Total solids at 110 C	35,680	3.56
Total organic matter (as determined by loss on ignition at 650 C)	30,760	3.07
Total inorganic matter	4920	0.49
Total sulfate (SO_4)	530	0.053
Suspended acid insoluble inorganic matter (sand, clay, etc.)	2570	0.26

*In all the references to percent organic concentrate solution used, it shall refer to the percentage of the above concentrate solution used in the mixing water and *not* to the percentage of organic matter used in the mixing water unless otherwise stated. The latter can be calculated by multiplying percent concentrate by 0.0307 or by referring to the conversion tables in Fig. 1 and 2.

The sulfate and suspended acid insoluble matter were determined from the material left after determination of organic matter. This ash was dissolved with dilute hydrochloric acid and filtered. The insoluble material from this treatment was weighed and reported as suspended acid insoluble matter. The filtrate was used to determine the sulfate content.

Analysis of organic concentrate solution

Just before use of this concentrate solution in making up test specimens, it was again thoroughly mixed and a representative sample taken. The analysis of the sample was made several days later at the time when about half of the test specimens were cast (this was not true for algae No. 12 which was mixed one month later). The analysis in Table 1 was representative of the series of tests.

TEST MIX PROPORTIONING

The proportioning of all trial mixes was based on the method and table on p. 131 of the *Concrete Manual* published by the U. S. Bureau of Reclamation, with quantities proportioned for a 150-lb batch. The proportions were also based on an assumed 1.5 percent of entrapped air, and a water content which after several trial batches was found to satisfy the given constants. By assuming incorrect air content no breach of standard procedure was committed, since these trial mixes (as proved later) could be adjusted to include the actual air content. This was possible because the amount of entrapped air did not affect the percentage of sand in the aggregate; it affected only the volume of the batch. Thus by reducing the water content and the volume of cement, sand, and coarse aggregate, and inserting the actual air volume, it was possible, after many trial calculations, to come up with a mix in which the proportions by weight were exactly the same as the proportions using the assumed air and water contents, and at the same time was the true design of the 1-cu yd mix as mixed.

For example, a mix was proportioned on the basis of a water content of 255 lb per cu yd, water-cement ratio 0.5, and 1.5 percent air. After mixing the batch the actual air content was 8 percent. Therefore, for correct proportioning of the given mix, the 8 percent was introduced, and by lowering the volume of cement, water, sand, and coarse aggregate correctly, a 1-cu yd mix was obtained that satisfied the original mix proportions and the new air content.

This greatly simplified the proportioning of mixes for all later concrete test batches, and meant one could proportion, in advance, a series of batches of varying water contents based on $1\frac{1}{2}$ percent air content and after a

few mixes get one to satisfy the required slump. In choosing the water content to be used, the 3 percent decrease in water content for each 1-in. excess slump was followed in spite of its inaccuracy. The reason for the inaccuracy was that the algae infested water increased the rate of change or slump for a given quantity of mixing water.

By observation it was found that a simple equation could be used to adjust the water content of the preliminary mix and so get a water content that would give the correct mix using the actual air content. After observation and trial the following equation was found to apply:

$$\text{Change in water content} = -2.62\ (\text{Actual air content, percent} - 1.5)$$

To substantiate that the above equation gave true 1-cu yd mixes without altering mix proportions, all batches were adjusted for the actual air content (Table 2).

DISCUSSION OF RESULTS

Sulfate content

At first it was thought that the sulfate content in the water might also cause a decrease in the strength of concrete since Duncairn Dam water is known to contain sulfates. But on analysis it was found to contain only 530 ppm in a 100 percent concentrate solution. Since in testing a 7½ percent concentrate solution was the maximum used, the greatest concentration of sulfates in mixing water was 40 ppm. Since Abrams[3] states that 0.5 percent sulfates or 5000 ppm causes a decrease in strength of 4 percent, the decrease in strength due to sulfates was neglected in these tests.

According to Abrams,[3] strength ratios for a series of tests should not fall below 85 percent due to the quality of mixing water. This would mean that the organic matter (algae) content should not exceed approximately 0.075–0.1 percent (Fig. 1) of the mixing water by weight. If the Joint Com-

TABLE 2—SUMMARY OF BATCHES TESTED

Mix	Organic concentrate in mixing water, percent	Actual slump, in.	Water content, lb		Actual air content, percent
			1½ percent air	Actual air	
		Group 1			
Algae 10	0 (control)	1½	300	298	2.2
Algae 8	1	1¾	300	297	2.6
Algae 7	3	1¼	255	243	6.0
Algae 5	5	1¾	245	229	7.9
Algae 12	7½	1½	247.5	233	7.0
		Group 2			
Algae 1	0 (control)	3	295	292	2.5
Algae 6	5	3½	255	233	9.8
Algae 9	7½	3¼	252	228	10.7
Algae 11	7½	3	252	230	10.5
		Group 3			
Algae 4	5	9	295	272	10.2
Algae 2	100	6¼	295	248	19.5
Algae 3	100	6¼	270	215	22.3

mittee Report were used, the organic matter (algae) should not exceed 0.05 percent by weight. Thus by selecting the median of these limits, the author believes that 0.075 percent would be quite allowable (when 85 percent of calculated strength was taken into consideration), but any excess of 0.1 percent organic matter (algae) in the mixing water should be regarded as excessive and consequently such water should be avoided.

Entrainment of air

After mixing the first few test batches, it was substantiated that algae caused air to be entrapped* into the concrete (Fig. 2). This was possibly due to surface active nature as stated in "Types of Algae," p. 334. The amount of air appeared to vary as a straight line with the organic (algae) concentration. As was seen by algae No. 2 and algae No. 3 (Table 2), the maximum amount of air it was possible to entrap was 22.3 percent, with an actual water content of 215 lb per cu yd and an actual organic (algae) concentration of 3.07 percent. Therefore, by further testing, one would get a curve in Fig. 2 between 0.25 percent and 3.07 percent organic matter (algae) concentration. Furthermore, it is believed that loss of strength due to organic particles reducing adhesion and the chemical reaction between the secretions and body fluids of the algae with the cement, thus reducing the bonding qualities, added to the decrease in strength of the concrete giving the over-all

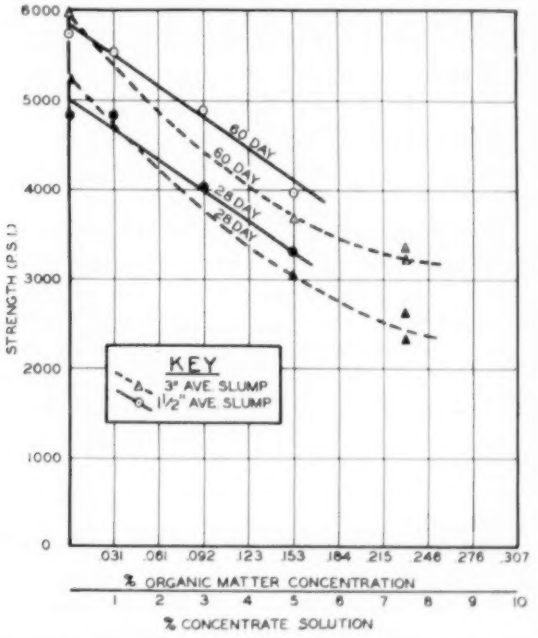

Fig. 1—Relation of organic matter concentration to concrete strength

*It could likely be that a percentage of the air in the cylinders was entrained air thus causing the decrease in water content. This research did not include microscopic study of the cylinders but such an investigation is being planned.

strength decrease shown. Some may wonder if the algae were dead and, if so, to what extent it would affect the results. The algae were dead because the solution was kept frozen until required and then thawed rapidly. This, it is believed, prevented decomposition and therefore the results should not have been affected to any extent.

Uniformity of concrete

Cylinders made from the different batches proved to be uniform, as is seen in Table 3. A coefficient of variation of 5 percent is considered good for laboratory cylinders. Most cylinders in this test did not exceed this. Those that varied exceeded it only slightly.

Slump

It was previously stated that a 1½-in. slump was used along with the 3-in. slump. In Fig. 1 and 2 both have approximately the same slope and almost the same strengths within the 85 percent strength range (suggested as the minimum for loss in strength due to impure water). Therefore, no harm seems to have been done by including the 1½-in. slump and none whatsoever from the point of view of establishing an approximate relation between strength loss and algae concentration. It is realized, of course, that the curves are approximate to a certain extent because of the limited number of tests made, and that further tests should be made for more accurate curves. From the objective viewpoint, these curves are believed to have provided a fairly accurate basis for determining the suitability of organic or algae infested water in mixing concrete.

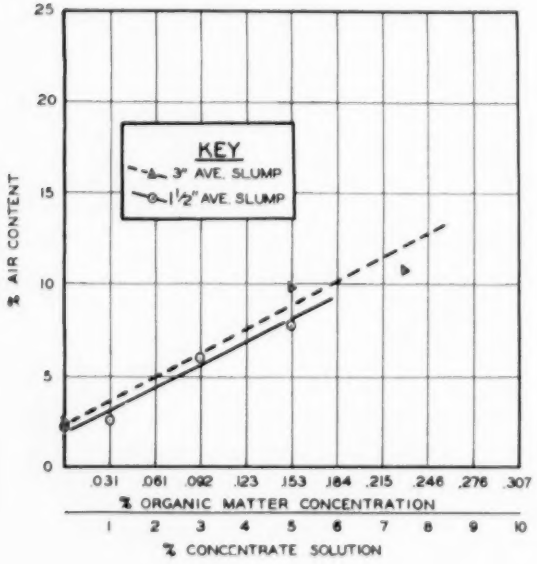

Fig. 2—Relation of organic matter concentration to air content of concrete mix

TABLE 3—SUMMARY OF COMPRESSIVE TESTS WITH COEFFICIENTS OF VARIAITON*

Laboratory batch	Organic matter, percent	Age, days	Cement content, lb per cu yd	Average strength,† psi	Coefficient of variation, percent
Group 1					
Algae 10	0	28	596	4832	0.6
	0	60	596	5742	2.2
Algae 8	0.031	28	594	4840	1.4
	0.031	60	594	4536	5.2
Algae 7	0.092	28	486	4038	4.4
	0.092	60	486	4897	2.2
Algae 5	0.153	28	458	3316	3.8
	0.153	60	458	3974	6.2
Algae 12	0.23	28	464	3326	0.6
	0.23	60	464	3930	2.8
Group 2					
Algae 1	0	28	584	5212	2.1
	0	60	584	5974	5.2
Algae 6	0.153	28	466	3042	2.2
	0.153	60	466	3676	1.4
Algae 9	0.23	28	456	2325	1.5
	0.23	60	456	3236	1.7
Algae 11	0.23	28	456	2628	0.9
	0.23	60	456	3346	1.9
Group 3					
Algae 4	0.153	28	544	2968	0.7
	0.153	60	544	3484	1.6
Algae 2	3.07	28	496	825	4.2
	3.07	60	496	1058	3.0
Algae 3	3.07	28	432	732	5.4
	3.07	60	432	811	8.0

*Calculated according to procedure on p. 173, *Concrete Manual*, U. S. Bureau of Reclamation.
†Average of five cylinders at both 28 and 60 days.

Variations from trend

With regard to the batch with algae No. 12, which seemed to differ in air content and strengths from the trend, it can be said that this was probably due to a change in mixing procedure and state and composition of organic matter since this batch was mixed one month later than the rest of the batches. Therefore, not too much importance was placed on the results and consequently the curves did not consider these points. Also, since that test did not fall within the 85 percentage range, its omission did not interfere with the basic information sought.

Strength-slump relationships

In Fig. 1, a strength decline for the 1½-in. slump batches was drawn as a straight line, while strength decline for the 3-in. slump batches was drawn parabolic. According to theory, greater slump would lessen strength of concrete but for some unknown reason the control cylinders for the 3-in. slump seem to have been stronger than for the 1½-in. slump. With further testing, the control batch of the 3-in. slump group would likely be found to be in error and fall below that of 1½-in. slump. The latter being true, a straight line relation would exist for the 3-in. slump group as for the 1½-in. slump group for the range of concentrations tested. However, since the results did not follow this, a parabolic relation had to be adopted until disproved.

SUGGESTIONS FOR FURTHER TESTING

1. It would be of interest to separate the loss in strength due to air content and the organic matter and secretions.

2. It would also be of interest to compare the loss in strength due to algae with other organic substances such as vegetable matter (roots, straw, etc.).

3. In further algae testing it would be advisable not to seal pails containing algae but leave them exposed to light and air so they will not die. If this is not feasible it might be even better to freeze the solution immediately. One should use the concentrate solution as soon as possible upon removal from the reservoir or water hole.

ACKNOWLEDGMENTS

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The author also expresses his gratitude to the College of Engineering, University of Saskatchewan and the PFRA Concrete Department for the permission to publish this abridged edition of his graduation thesis.

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Title No. 51-17

Observations on the Resistance of Concrete to Freezing and Thawing*

By HUBERT WOODS†

SYNOPSIS

A survey and analysis of the more important investigations on freezing and thawing of concrete show that significant general conclusions can be drawn. Some of these conclusions are presented in a manner designed to show their reasonableness, but without detailed evidence.

During the past 10 or 15 years many investigations have been carried out on the question of durability of concrete under freezing and thawing. The principal objective of much of this research has been the determination of the comparative behavior of specimens of concrete made from various cements and aggregates, with various mix proportions, water-cement ratios, and air contents; cured in specific ways for various lengths of time; and subjected to various freezing and thawing tests. Some research has been of a fundamental nature, having as an object the determination of the conditions and mechanism of deterioration and the role of air entrainment.

In view of the wide variety of concrete materials, mix proportions, curing procedures, and test conditions employed, with a consequent wide variety of results obtained, it is not surprising that a considerable amount of uncertainty exists as to the meaning of the published results.

A survey and analysis of the more important investigations on the subject show, however, that significant general conclusions can be drawn. It is the purpose of this paper to present some of these conclusions in a manner designed to show their reasonableness, but without detailed evidence. What is intended is simply a concise presentation, which the reader, if he wishes, can check against published data.‡ No originality is claimed for any of the views or conclusions here stated.

Dry hardened concrete, or concrete in which the hardened cement paste and the aggregate are both below some critical degree of saturation by water, is highly resistant to freezing and thawing in nature. Such concrete could probably be damaged by thermal stresses set up within the concrete due to large and rapid temperature changes, even if the lower temperature were above the normal freezing point; however, there is little evidence to show that damage from this cause alone is normally met in natural exposure. Thermal

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‡A brief bibliography is appended.

stresses might contribute to, or accelerate, a deterioration resulting from other causes such as alkali-aggregate reaction. That subject is beyond the scope of the present discussion.

Water-soaked, hardened concrete may be damaged by freezing and thawing, and the locus of damage may be the hardened cement paste, or the aggregate, or both. In any of these cases, the damage arises basically from the fact that the process of freezing water within the paste or the aggregate gives rise to internal pressures which may be high enough to exceed the strength of the paste or of the aggregate. The aggregate particles and the hard cement paste may appear to be solid bodies; they are so only superficially. Some aggregates are capable of absorbing several percent of water. The hard cement paste may include, to the extent of a fifth of its own volume, capillary space capable of holding water. The capillary structure of cement paste is very fine, beyond the limit of resolution of light microscopes. Because of this, and in view of the large volume of freezable water it may hold, it is not surprising that hardened cement paste may be damaged by freezing and thawing.

Damaging pressure in hardened cement paste may conceivably arise in any of several ways. First, there may not be sufficient empty space available to accommodate the additional solid volume produced when the contained water freezes. This possibility has given rise to the "degree of saturation" theory of frost damage. Second, the pressure may be hydraulic, due to the resistance of the fine-grained capillary structure to the flow of water displaced by the advancing ice front. Third, the pressure may be caused by the growth of ice bodies extracting water from the surrounding gel. Present evidence suggests that both of the last two mechanisms are of importance in cement paste, whereas the first may be of primary importance in stones used as aggregate.

It may seem that although the purpose of this paper is ostensibly to discuss the resistance of *concrete* to freezing and thawing, attention has been diverted by a discussion of cement paste. Actually this is not a diversion. It is essential to an understanding of concrete to recognize that this material is not a homogenous solid and to inquire into the properties of its component parts and their mutual relationships. Research has made it evident that the basic properties of the hardened paste are of prime importance in providing an understanding of the behavior of concrete.

Reference was made to the possibility of developing damaging pressures inside stones or hardened cement paste when water contained therein is frozen. If the degree of saturation is below some critical proportion, about 90 percent, there will ordinarily be present enough air-filled capillary space to accommodate the water displaced by the ice being formed. However, in the fine capillary structure of cement paste, this space must be everywhere close to the points from which water is moving to avoid destructive pressure. The paste in non-air-entrained concrete may, and usually does, contain large natural air voids in sufficient total volume to contain the water displaced during the freezing process, yet such paste may be dilated and damaged by

rapid freezing when saturated because the natural air voids are too far apart to be effective. This statement is no longer conjectural; it has been confirmed experimentally. It has also been shown that freezing pastes of the same kind does not cause dilation or damage when such pastes contain numerous air bubbles. It should be made clear that in this discussion the term "saturation" is applied to the cement paste exclusive of the entrapped or entrained air voids. These air voids, being much larger than the capillary spaces, do not ordinarily become filled with water.

The vital role of entrained air now becomes evident. It is to provide in the hardened paste a multitude of air voids separated from one another by only a few thousandths of an inch, so that excess water displaced during the freezing process has only a short distance to move before finding relief space. The extruded water, frozen in the air bubbles, is able to extract additional water from the surrounding paste and thus prevent the growth of ice bodies in the paste. In these ways, pressure is limited, and if the air voids are close enough together, the pressure will not exceed the strength of the paste. It is now well established that the air-entraining agent performs its valuable role by means of the air it entrains, and not by its presence *per se*, nor by any effect it may have on the physical properties of the cement paste.

The methods currently used to test the frost resistance of concrete need to be re-examined critically in the light of present knowledge of the mechanism of deterioration. It seems entirely possible that accelerated freezing and thawing tests could be devised which would result in the rapid destruction of air-entrained concrete which in nature would prove highly resistant. Indeed, it may be questioned whether some of the high-speed tests now in use yield results from which valid predictions can be made of behavior in natural weathering. It is not essential that laboratory tests exactly reproduce natural processes, but it is desirable that laboratory tests produce information useful for prediction of behavior in such processes.

The question naturally arises whether one or another composition of cement, or a cement ground to one or another fineness, has outstanding advantages in resistance to freezing and thawing of well-cured concrete. The evidence is definitely to the contrary. Experiments with non-air-entrained concretes of commonly used water-cement ratio, using cements of various types, ground to a wide range of finenesses, show that all are vulnerable to freezing and thawing when water soaked. Fundamental studies of hardened cement pastes made from various types of cement show that at normal water-cement ratios, and with adequate curing, the physical structures are similar. To be more specific, all these pastes are capable of holding more freezable water than can be frozen without damage, unless the internal pressure which would otherwise result is limited by some means. Fifteen years of experience in field and laboratory have shown that entrained air is exceedingly effective in avoiding this damage, increasing the durability of otherwise good concrete by more than tenfold. Thus, field experience, laboratory ex-

periment, and theory show that entrained air in proper amount and distribution is essential to produce concrete able to resist repeated cycles of freezing and thawing while water soaked.

The primary function of entrained air is therefore to protect the cement paste in concrete from the ravages of freezing and thawing. Entrained air in the cement paste cannot be expected to make bad aggregate good, or to eliminate the need for proper mix proportioning or construction practices. It would indeed be unfortunate if air entrainment were to be considered a cure-all.

Nothing in the foregoing discussion should be construed to minimize the importance of cement composition or fineness with respect to other matters. It is common knowledge that the rate of hardening, and possibly the maximum strength attained, depends upon cement composition and fineness. The manner in which concrete resists attack of aggressive solutions, such as those of alkali sulfates, is known to depend on cement composition. Shrinkage on drying and expansion on wetting depend on cement composition. The amount of heat liberated by cement during hardening also depends upon cement composition and fineness, a matter of importance to designers of massive concrete structures. It may well be that other important properties of concrete are influenced, perhaps largely, by these factors.

Research has suggested an alternative to entrained air, as a means of protecting the hardened cement paste, when circumstances permit. If the original water-cement ratio does not exceed about 0.4 by weight, the resulting hardened paste when well cured will contain no water that will freeze under conditions of natural exposure. Some of the water is chemically combined and the rest is strongly adsorbed. Even when saturated, such paste should be highly resistant to normal freezing and thawing. Further experimental verification of this indication is desirable. A water-cement ratio of 0.4 by weight is about 4½ gal. per sack, and thus somewhat lower than that ordinarily used.

As the water-cement ratio is increased above about 0.4, the capacity of saturated cement paste for freezable water increases regularly. Also, permeability increases. Thus concrete of high water-cement ratio is capable of holding much freezable water, and furthermore the paste it contains is more likely to become saturated in natural exposures owing to the higher permeability. For these reasons, such concrete if made without entrained air is more likely to deteriorate under freezing and thawing, and experience bears this out.

It is widely recognized that air-entrained concrete is resistant to surface scaling consequent upon the application of salts for ice removal. Research has not yet provided a clear understanding of the scaling phenomenon, nor of the role of entrained air in minimizing it. The literature reveals several possible explanations, but none has been proved. The problem is likely one of physics or physical chemistry, as so many problems involving the properties of concrete seem to be.

In summary, the following conclusions can be stated with reasonable assurance:

1. Concrete of normal water-cement ratio, without entrained air, is vulnerable to freezing and thawing when water soaked.

2. The locus of damage may be the aggregate, or the hardened cement paste, or both. The damage is a result of excessive internal pressure developed during the freezing of water in the aggregate or paste.

3. No portland cement clinker composition or cement fineness within the present or past commercial range has any significant advantage over any other with respect to the resistance of well-cured concrete to freezing and thawing when water soaked.

4. The primary function of entrained air is to protect the hardened cement paste from destructive pressure arising from the freezing of water therein. Fifteen years of practice show that it does this exceedingly well. However, entrained air should not be considered a substitute for durable aggregate, proper design, or good construction practice.

5. A possible alternative to entrained air may be the use of a low water-cement ratio, below about 0.4 by weight. This needs confirmation.

6. Practice shows that concrete containing a proper amount of well-distributed entrained air is resistant to surface scaling consequent upon the application of salts for ice removal. The mechanisms of scaling, and of the protective effects of entrained air, have not been definitely established.

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Title No. 51-18

Some Failures of Reinforced Concrete Storage Bins*

By LOUIS E. VANDEGRIFT†

SYNOPSIS

Some failures of reinforced concrete storage bins due to improper reinforcing of the fillets at the interstice bin corners are illustrated. Simple model tests demonstrate the need for steel at these points.

CASE HISTORIES

The use of reinforced concrete for the construction of multiple cell grain storage bins offers a durable and economical structure. The successful use of this material, however, requires that the designer be well-versed in effective methods of joining intersecting walls in such a manner that the stresses will be carried safely around the angles of intersection. No attempt is made in this discussion to offer a mathematical solution to the structural problems involved, especially with regard to groups of circular bins joined by short straight connecting walls.

In the case of multiple cylindrical bins where the sides of the closely spaced bins are connected with short walls, the space bounded by the exterior of the walls of four adjacent circular bins and the short connecting walls is also used for storage. This space is called the "interstice." Often the capacity of this space offers storage capacity equal to or greater than the volume of one of the cylindrical bins. In the author's opinion, the size of the interstice should be kept to a minimum to reduce the effects illustrated in Fig. 1. Frequently, however, smaller storage bins are desired; arrangement of the interstice to provide smaller bins is not difficult.

Fig. 1 shows a cardboard model of a group of eight circular bins and three interstice bins. The group of four circular bins on the left were filled with dry sand after which the interstice bin was loaded. The result was a stable condition. When the interstice bin on the right was loaded while the adjacent circular bins remained empty, the qualitative effect was as shown. The slenderness of the wall sections in the model accentuates the results of the bending influences for better visualization. It is clear that severe bending was caused at the intersection of the circular walls and the short connecting walls. There would also be direct tension in the short connecting walls.

The practice of using the elevator without regard to the order of bin loading

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Fig. 1—Cardboard model of typical group of circular storage bins showing two loading conditions

is widespread. It is the duty of the engineer to his client to design and construct these structures so that his client can so use the bins. Occasionally, however, adequate provision for effective tying together of the walls is not made. A result of such failure is shown in Fig. 2 and 3. Fig. 2 shows a structure which was composed of four adjacent bins and one enclosed interstice. The interstice was loaded while the adjacent bins were empty. There was high tensile stress due to bending, plus direct tension at the intersection of circular and plane surfaces of the walls inside of the interstice.

The dimensions of the bins in Fig. 2 are not available, but the failure occurred by the missing cylindrical bin being ripped loose and shoved sidewise by the wheat stored in the interstice when the collapsed bin was empty.

The loading of an interstice bin when adjacent bins are empty creates a structural problem of considerable complexity. Because of the shape of the interstice there will be considerable variation in the pressure of grain on the interstice walls at any given level. Due to the type of bending indicated

Fig. 2 (left)—Failure of storage bin due to inadequate provision for effectively tying the walls together. The interstice was loaded while the adjacent bins were empty. Fig. 3 (right)—Failure due to faulty design of the junction of the circular bin walls and the short connecting walls. The interstice was loaded while adjacent bins were empty. Note distortion of cylinder wall at right

qualitatively in Fig. 1 there will be translation and rotation of the intersection of the cylinder walls and the straight short connecting walls. The problem is further complicated by the restraining action of the foundation slab on the rotation and translation. It is easily seen that when all of the interstice bins are loaded while the adjacent circular bins are empty, the condition of maximum bending exists in the circular bin walls at their intersection with the short straight walls which separate the interstice bins. It is suggested that the safest structure of this type will result from the smallest possible size interstice. This will result in a smaller hydraulic radius of the interstice cross section. Since the normal grain pressure increases in some manner as the hydraulic radius increases, the smallest interstice area seems desirable from a safety standpoint.

Fig. 3 shows a close-up of a ruptured bin in a structure that was composed of 14 circular bins and six interstice bins. The circular bins are 23 ft inside diameter and the bin walls are 7 in. thick. The hoop reinforcement is #5 bars which are spaced 7 in. on centers. These bars are very close to the outside face of the cylinder walls. The bins are approximately 130 ft high. The circular bins are 5 ft clear distance apart. At the time of failure, an interstice bin was loaded with about 40,000 bushels of wheat while the four adjacent circular bins were empty. The failure appeared to be the result of faulty design of the junction of the circular bin walls and the short connecting walls. The horizontal cross section of this structure is geometrically similar to the model in Fig. 1. Fig. 3 shows that the distortion of that part of the cylinder wall next to the interstice is similar to the distortion illustrated in the cardboard model in Fig. 1.

A study of Fig. 3 will lead one to the conclusion that this structure would have been considerably stronger if the following had been provided: (1) main hoop reinforcing in the vertical cylinder walls placed so that the steel was near the outer surface of the cylinder walls at the junction of the short connecting walls and near the inside face of the cylinder walls at the crown of sections that function as an arch; (2) short straight connecting walls placed where the round bins were closest to each other; (3) reinforced fillets, at least 10 x 10 in., should have been provided at the intersection of all vertical bin walls; and (4) proper anchorage of steel in intersecting walls to prevent ripping apart of the walls.

There are relatively few failures of circular bins similar to those shown in Fig. 3, especially when the number of failures is compared to the total number of structures in existence. However, since this structure failed structurally after many years of service, and since it has a bad history, it seems important to point out weaknesses that were apparent from an examination of the structure after failure. It is felt that many of the structures now in use are working outside of the factor of safety, and that the inherent toughness of reinforced concrete should be given credit for the large percentage of successfully functioning structures.

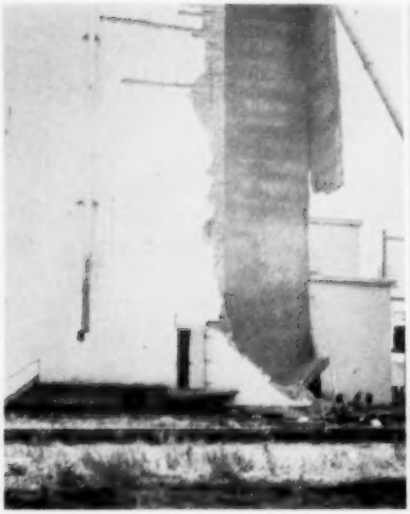

Fig. 4 (left)—Structural failure of rectangular storage bins—corner fillets at wall intersections were only a few inches wide. Fig. 5 (right)—A failure of rectangular storage bins where corner fillets at wall intersections were unreinforced

Fig. 4 and 5 show two different structural failures occurring in rectangular storage bins. An examination of the collapsed structures revealed that little sound consideration had been given to the design of the wall intersections. In the case shown in Fig. 5, unreinforced fillets had been built in the corners of the rectangular bins. The surface of these fillets made a 45-deg. angle in the corners and were about 12 x 12 in. In the case shown in Fig. 4, the corner fillets were only a few inches wide.

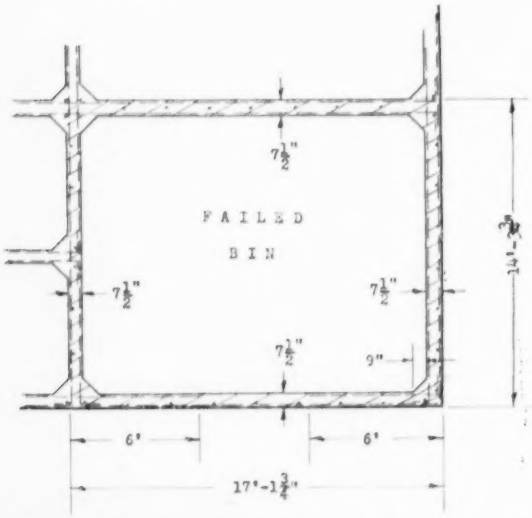

Fig. 6—General scheme of reinforcing for rectangular structures

The structure shown in Fig. 5 had apparently been designed so that the corners including the fillet areas would act as columns from the footing level to about 9 ft above the hopper area. The fillet areas, below the region of failure in the bin that failed, had been provided with triangular hoop steel which was apparently meant to act as column reinforcement. The failure began a

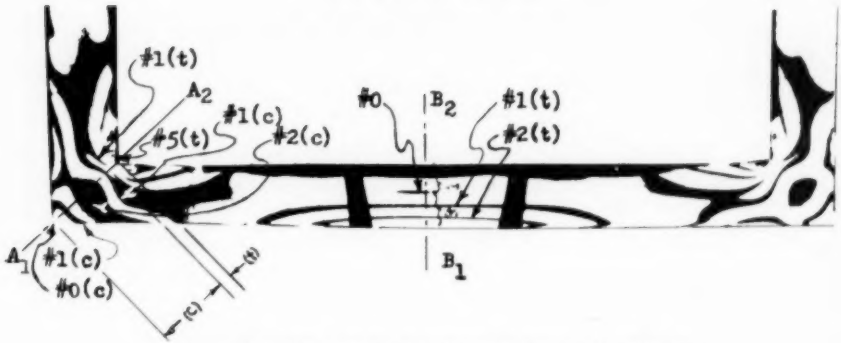

Fig. 7—Photoelastic study of rectangular frame without fillets

few feet above the level where use of the triangular hoops was discontinued. This agrees with the model test results which are described later. The hoops acted as fillet reinforcement.

In the structure shown in Fig. 4, where there was no fillet reinforcement, the side walls were ripped off down to the hopper level. In addition to the absence of the fillet steel, poor provision had been made for proper anchorage of the steel which was used at the corners in these cases.

The concrete in all of the structures was of good quality. One of the structures with circular bins was about 35 years old at time of failure, but examination revealed successive displacements and ineffective attempts at repair. All of the rectangular bins were only a few months old when they failed, but they had been filled with grain. In every instance of failure, it appeared to be a case of improper use of good material.

PHOTOELASTIC STUDIES

The general scheme of reinforcing for all the rectangular structures being considered herein is shown in Fig. 6. The structure shown in Fig. 4 was even less effectively reinforced than the structure shown in Fig. 5. There is no reinforcement across the face of the fillets. Since negative bending at the corners will cause tension in the face of the fillet concrete, cracking will start where there is a high concentration of stress and the crack will extend to the steel. A stress concentration will then exist similar to that shown by the photoelastic study in Fig. 7.

Fig. 7 shows a photograph of the fringes developed by passing polarized light through a rectangular plastic frame which was loaded to simulate the outward pressure of granular material. One side and part of the two adjacent sides are shown. It is apparent that there is a severe concentration of stresses at the reentrant corners which are in bending tension. Fig. 8 shows how the fringes develop when fillets are provided, the loading being the same as that used on the model shown in Fig. 7. Stresses at the corners due to bending tension are not nearly so severe as shown in Fig. 7, although stresses do concentrate at the intersection of the planes of the inside wall surfaces.

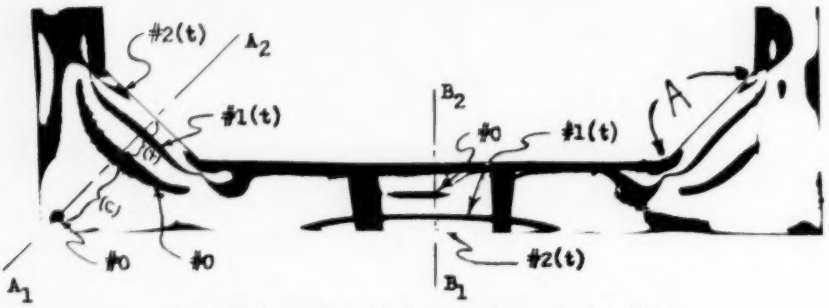

Fig. 8—Photoelastic study of rectangular frames with fillets

Reference to the photoelastic model shown in Fig. 8 will show that cracking in the concrete will probably start at point A and will progress directly to the intersection of the negative steel at the corner. The stress condition will then be qualitatively represented by the photoelastic fringe pattern shown in Fig. 7. The steel will be highly stressed and the effective depth of the reinforced concrete section at the corner will be small, as will be demonstrated later. The area of concrete at the outside corner of the frame will carry little stress. The fringe photographs indicated how failure would occur. The photoelastic studies suggested that tests on model reinforced mortar continuous frames would be helpful.

REINFORCED MORTAR MODELS

The plan of a one-twelfth scale model is shown in Fig. 9. To include two layers of 14-gage steel wire the model frame was made 1.5 in. high. This represents a frame section 14 ft x 16 ft x 1 ft 6 in. high, the side slabs being $7\frac{1}{2}$ in. thick, the fillets are 12 x 12 in.

The models were made of mortar mixed in the following proportion by weight: 1 part water, 1.85 cement, and 4.80 parts sand. The sand was a well graded, exceptionally clean sand passing a screen with openings of 0.065 in. This size permitted the mortar to pass freely around the reinforcement and between the reinforcement and forms. The cement was a high early strength cement.

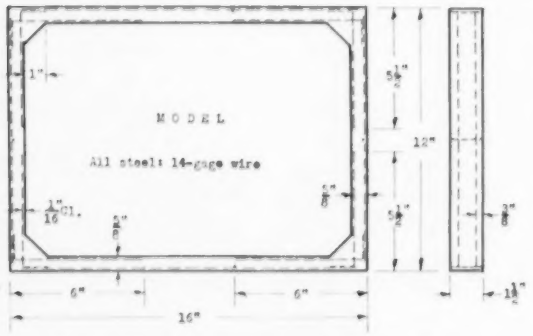

Fig. 9—Plan of scale model of continuous frame

A hard grade of 14-gage steel wire was used as reinforcement in part of the models. In the remainder a softer wire was used. Table 1 gives the physical properties of the wire.

TABLE 1—PHYSICAL PROPERTIES OF WIRE IN MODELS

Wire	Diameter, in.	Area, sq in.	Elastic limit, psi	Tensile strength, psi	Elongation, percent	Area reduction, percent
Hard	0.08	0.00503	99,480	125,530	1.75 in 8 in.	32
Annealed	0.08	0.00503	50,000	80,000	10 in 2 in.	60

The forms for the models were made of sheet steel formed in a sheet metal brake. The forms were carefully fitted and bolted to a plate. The models cast in these forms were very satisfactory and the forms were used repeatedly. After each use they were carefully cleaned and oiled for reuse.

After the mortar was mixed, it was allowed to set, covered with wet burlap for 1 hr before placing in the forms. During this hour the mortar was thoroughly remixed every 15 min. This operation was meant to reduce the shrinkage that would occur during setting. At the end of 1 hr the mortar was carefully rodded into the forms after which the plate to which the forms were bolted was vibrated with a small hand pneumatic vibrator having a frequency of 7000 small amplitude oscillations per minute.

Immediately following the placing and vibrating of the mortar in the forms, the models were placed in a warm vapor bath at 100 F for 24 hr. At the end of 24 hr the forms were removed carefully and the specimens were placed in standard fog room until tested. The damp storage was in excess of 28 days.

The method of loading the models is shown in Fig. 10. The loads were applied to the strips of wood by small tubes. The load was transmitted to the model through a layer of sponge rubber. The purpose was to apply a load approximating a uniform load.

Fig. 11 shows a model that has been tested to failure; in the foreground is a reinforcement cage like the one used in the model. It should be noted that steel for reinforcing across the fillets has been provided. This fillet steel extends to the outside face of the model, thence parallel to the outside face until sufficient length has been provided to develop the strength of the wire in bond. The photoelastic fringes in Fig. 7 and 8 indicate that the fillet steel must check the opening of cracks which appear in the intersection of the sur-

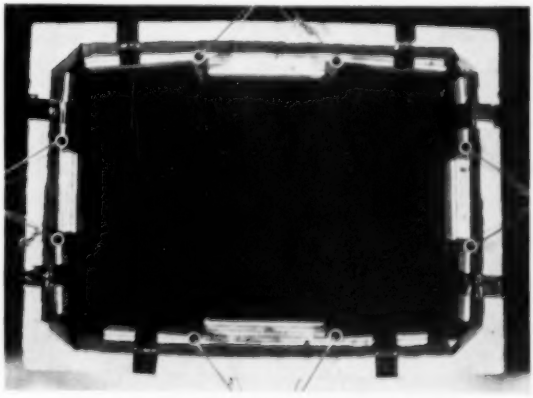

Fig. 10—Method of loading models

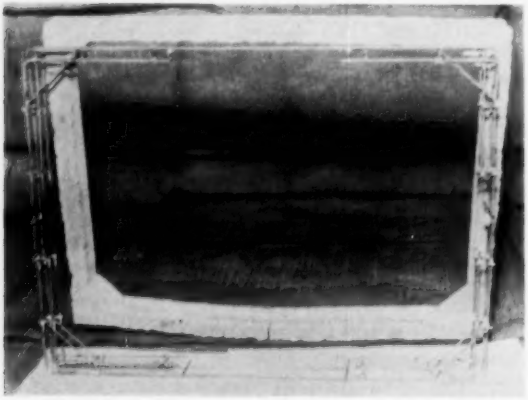

Fig. 11—Model with reinforced fillets tested to failure. Reinforcement cage is in foreground

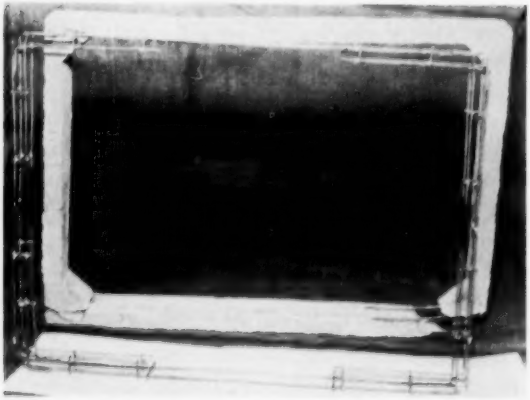

Fig. 12—Model with no fillet reinforcement tested to failure. In this case the load was allowed to remain until one of the corners broke away. Reinforcement cage is in foreground

face of the fillet and the inside surface of the bin walls.

Fig. 12 shows a second model that has been tested to failure. The load was allowed to remain until one of the corners broke away. In this model no fillet reinforcement was used. When Fig. 12 is compared to the photoelastic fringe concentrations in Fig. 8, it is seen that cracks should start in the corners where the plane of the fillet surface intersects the inside plane of the walls.

Table 2 gives the data on the breaking loads for the various tests. The percent of effectiveness is obtained by comparing the total breaking load on the sides of the model which was not provided with fillet steel to the total breaking load on the sides of the companion model which had been provided with fillet steel.

TABLE 2—SUMMARY OF TEST DATA ON MODELS

Group	Set	Mortar strength at time of testing, psi†	Fillet steel	Load on short spans, lb	Load on long spans, lb	Ratio of short span to long span loads, percent	Effectiveness without fillet steel, percent
A*	1	3200	No	186	243	76.5	55.8
			Yes	339	429	77.8	
	2	3800	No	147	184	79.7	49.2
			Yes	227	447	50.8	
	3	4900	No	189	246	76.8	45.6
			Yes	399	555	71.9	
B*	1	3000	No	174	236	73.7	62.9
			Yes	280	372	75.3	
	2	4180	No	240	318	75.5	62.6
			Yes	378	513	73.7	
	3	4740	No	199	267	74.5	48.8
			Yes	408	549	74.3	

*Group A made with hard wire.
*Group B made with annealed wire.
†Test cylinders were 2 x 4 in.

Title No. 51-19

Static and Fatigue Tests on Partially Prestressed Concrete Constructions*

By PAUL WILLIAM ABELES†

SYNOPSIS

Discusses an investigation of the essential features of partial prestressing. Static and fatigue tests as well as routine acceptance tests are described.

A few of the conclusions drawn are: (1) Cracks should not form in acceptance tests under bending tensile stress of 750 to 800 psi for pre-tensioned wires or 650 psi for post-tensioned wires. (2) In the former case no cracks should form under fatigue loading corresponding to a bending tensile stress of 650 psi. (3) When cracks have occurred and fatigue loading takes place, up to a bending tensile stress of 650 psi, the cracks disappear on removal of the load and no noticeable deformation remains. (5) Several million repetitions of the fatigue load do not affect the subsequent static failure load.

INTRODUCTION

An earlier paper[1] explained the difference between full and partial prestressing (concrete bending tensile stresses under working load must not occur with full prestressing but are permitted with partial prestressing). Since then, the author has made many tests to investigate the behavior of partially prestressed concrete under static and fatigue conditions. Permanent freedom from cracks can be attained in spite of limited, though not inconsiderable, tensile stresses under working load. Static failure tests were carried out in England in 1949 and 1951, and fatigue tests were conducted at Liége, Belgium, in 1951, which will be described. In the practical application of this system, many routine acceptance tests were made which will also be mentioned.

STATIC FAILURE TESTS—1949 AND 1951

In 1946, the author conducted preliminary tests on prestressed railway ties,[2] loaded as simply-supported beams, when the extraordinary resilience of prestressed concrete was ascertained. Even after a loading as high as 85 percent of the failure load, when considerable deformation and heavy cracking had occurred, the cracks closed up entirely and became invisible on removal of the load. Based on this resilience and on the fact that cracks became visible at first loading at a tensile stress of 1000–1100 psi, partial prestressing

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†Member American Concrete Institute, Consulting Engineer, London, England.

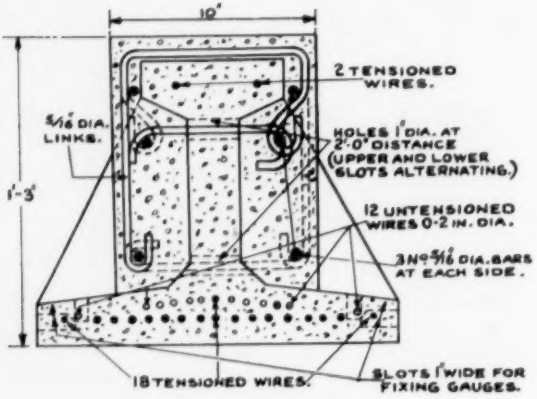

10"
2 TENSIONED WIRES.
5/16 DIA. LINKS.
HOLES 1" DIA. AT 2'-0" DISTANCE (UPPER AND LOWER SLOTS ALTERNATING.)
1'-3"
12 UNTENSIONED WIRES 0·2 IN. DIA.
3 No. 5/16 DIA. BARS AT EACH SIDE.
18 TENSIONED WIRES.
SLOTS 1" WIDE FOR FIXING GAUGES.

Fig. 1—Cross section of 1949 test beams

was accepted in 1948 by the Civil Engineer's Department, Eastern Region, British Railways, as suitable for a number of high road bridges for which reduction of construction depth was essential.

Full-size beams for bridge construction[3] were tested in 1949.[4] These 30 ft long composite members contained tensioned and nontensioned wires (see cross section in Fig. 1) to provide partial prestressing. It was essential to investigate such construction before practical application, since it was considered possible that permanent elongation might develop in the nontensioned wires, and consequently cracks would not close on removal of load. However, these 1949 tests proved that the behavior of composite beams with tensioned and nontensioned wires was quite similar to that of precast beams containing tensioned wires only. On removal of loads of approximately 80 percent of the failure load, the cracks closed completely; from this it can be concluded that the permanent elongation in the nontensioned steel was so small as to be unnoticeable and did not prevent complete closing of the cracks. The nontensioned wires were placed in "twin-twisted" pairs, with a pitch of 1 ft, to improve mechanical bond.

The stress-strain diagram of the wire (dotted line in Fig. 2) must be transferred in accordance with the stresses effective at the commencement of loading to obtain the actual curves for the tensioned and nontensioned wires. Assuming the effective prestress in the tensioned reinforcement as 110,000 psi, indicated in Fig. 2 by A, then the curve exceeding this stress must be transferred parallel to the origin O_t. The nontensioned wire is compressed somewhat during prestressing. If this compressive stress is taken as 10,000 psi, indicated in the extended dotted curve by A', then this curve has to be moved parallel to a new origin at O_u. If tensioned and nontensioned wires are in the same plane, the elongations of both must be the same under load. Consequently, in a cracked section in which such elongations occur, the stresses in the tensioned and nontensioned wires will differ only slightly when the stresses approach ultimate strength. This explains why the ultimate strength of both the tensioned and nontensioned wire is reached in most cases

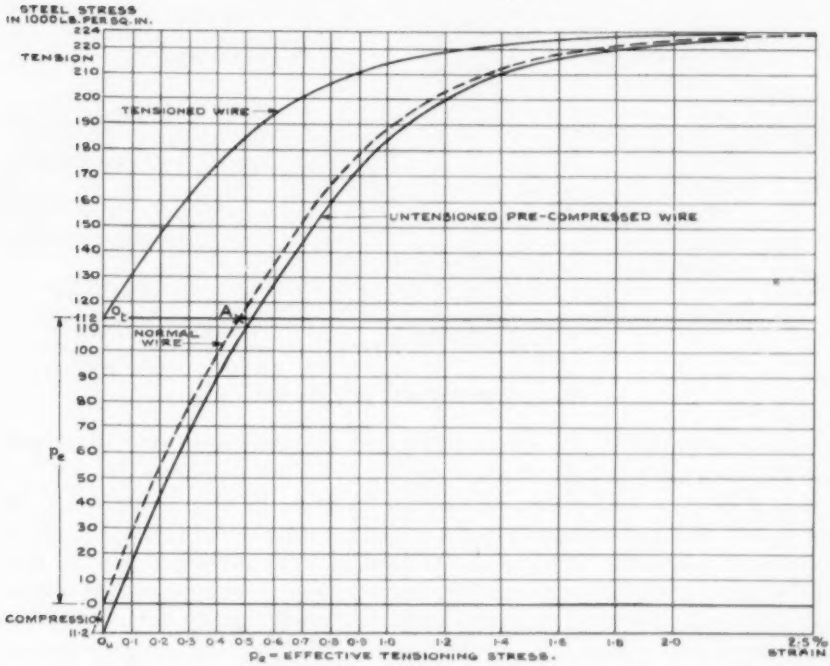


Fig. 2—Stress-strain diagram of 0.2-in. diameter tensioned and nontensioned wire

under ultimate load conditions. In under-reinforced sections, whether the full strength of the steel is reached or not depends entirely on the efficiency of the bond of the wire, *i.e.*, on its surface conditions and on the strength of the concrete.

The 1949 beam tests clearly proved the satisfactory cooperation of tensioned and nontensioned wires. However, these test beams did not represent typical bridge construction since the nonprestressed concrete was added outside the precast member (Fig. 1); in bridges the additional concrete is placed between two adjacent beams. Consequently, bond failure between prestressed and additional concrete occurred in two tests at approximately 93 and 95 percent of the load at which ultimate failure was expected.

In 1951, new tests were carried out on 20 ft long composite slabs with a cross section as shown in Fig. 3, where the additional concrete was placed between the precast beams as in actual bridge construction. These tests proved that the calculated ultimate load was fully reached; at the same time the other properties determined in the 1949 tests were confirmed. Two composite slabs were tested. In one the additional concrete was the same as the precast concrete (mix by weight, 1: 1.25: 4; water-cement ratio, 0.40; prism strength at 28 days, 4300 psi); in the second slab the added concrete had a much higher water-cement ratio, half the cement with the same water content. However, there was only a slight difference in the strength of the

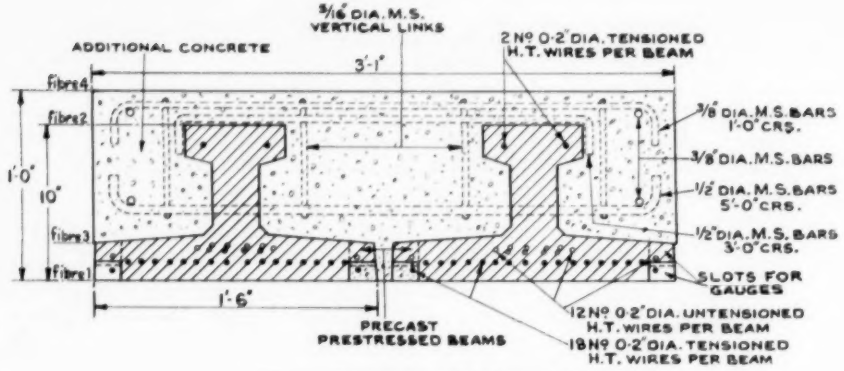


Fig. 3—Cross section of 1951 test slabs

two types of concrete, the reason being that the same aggregates were used and the concrete was well vibrated, thus driving out surplus water.

Fig. 4 shows a load-deflection diagram (loading in English tons) for the test slab in which the strength of the added concrete was the same as that in the precast concrete. Three cycles of loading were applied before the final failure loading. The curve for the first cycle is shown twice (the dotted curves show the deflection on a large scale). It is seen that the deformation line is curved before cracks were noticeable. In addition, the concrete stresses calculated for a homogeneous section are shown for fiber 1 (bottom fiber of slab) and fiber 3 (bottom fiber of added concrete). It is seen that the load at which cracks became visible exceeded a concrete bending tensile stress of approximately 1000 psi, as ascertained in previous tests. Also, the calculated average steel stresses, in percent, related to $f_{s\ ult}$, the steel stress at failure, are plotted. They were computed on the assumption that in this under-reinforced section the ultimate tensile force equals the product of the sectional steel area and the strength of wire. The ultimate stress distribution in the concrete compression zone is approximated by a rectangle with a maximum stress equaling the concrete prism or cylinder strength. In the special case only, 98.8 percent of the calculated maximum was obtained, which may be explained by the lower steel strength. Fig. 5 illustrates the deformation of the slab. It can also be realized from Fig. 5 that nonuniform loading may take place with such an arrangement, with the consequence that one side is strained to a much greater extent than the other. This will be discussed later when the third fatigue test is considered.

FATIGUE TESTS—1951

Three composite slabs of the type shown in Fig. 3 were tested for fatigue resistance at the laboratories of Prof. F. Campus, Liége, Belgium. The first two slabs were of the same type of concrete as the slab tested in the static loading test illustrated in Fig. 4. Both the precast and added concrete were made from a 1:1.25:2.5 mix by weight using high early strength portland

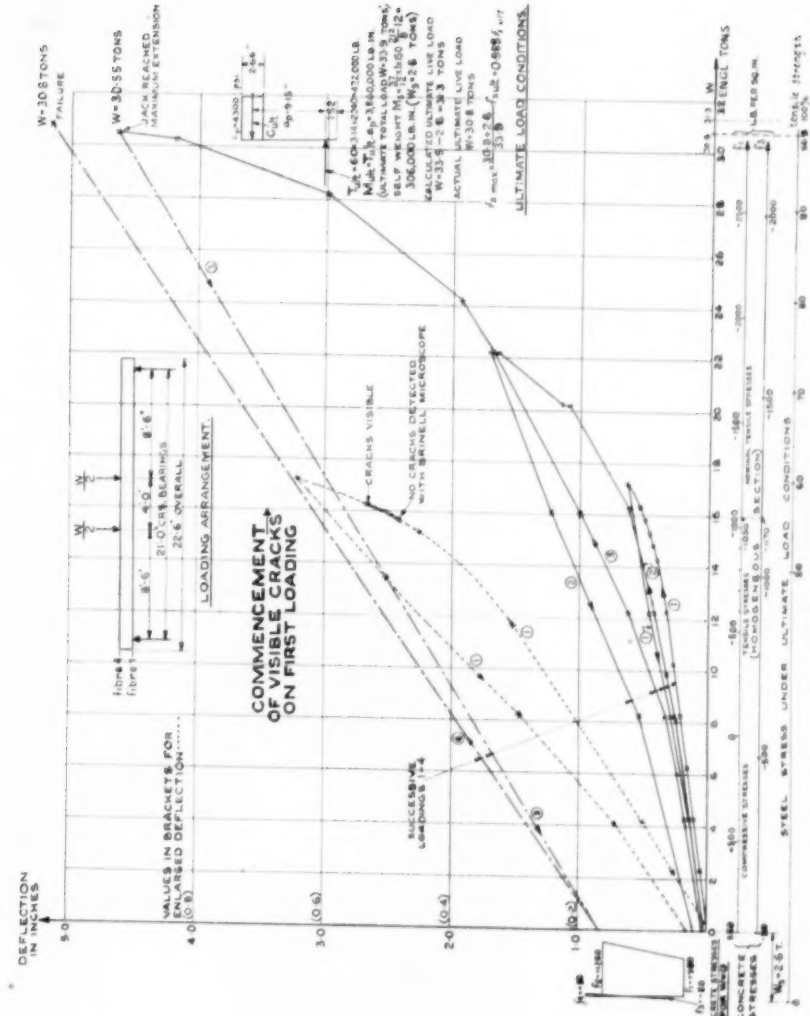


Fig. 4—Load-deflection diagram for 1951 static load

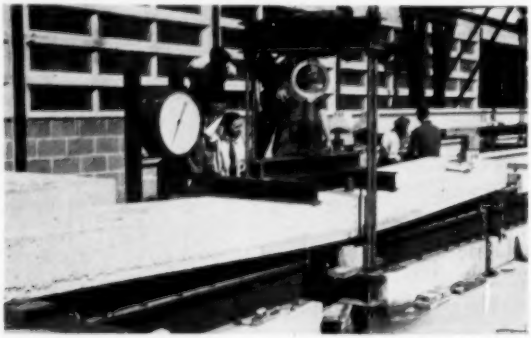


Fig. 5—Deformation of 1951 test slab under static load

cement with a water-cement ratio of 0.39. The prism strength of the concrete at the dates of the failure test of the two slabs were 5850 and 6150 psi (respective ages, 60 and 74 days). The cold drawn wire had an ultimate strength varying between 200,000 and 210,000 psi, while the strength of the wire used for the static test (see Fig. 4) was slightly less than 224,000 psi. In addition to these two slabs, tested in June and July, 1951, another slab containing concrete made with a Belgian sulfate-resistant metallurgical cement and with slightly different prestress due to different age was tested in November, 1951.

Fig. 6 shows the first two slabs being tested with the slabs placed in reverse position and the load applied from underneath. This allowed easier and more

Fig. 6—Testing arrangement for fatigue test at Liege, 1951. Slabs were loaded from the bottom

thorough observation of the cracks in the tensile zone. Previous to testing, these two slabs had been deliberately loaded statically until a number of cracks developed, so as to carry out the fatigue tests in a cracked state. Such a test condition was considered essential because in a bridge cracks may develop due to an unforeseen higher loading, and then fatigue loading may take place under which these cracks open and close repeatedly. Since the bridges were designed for a permissible bending tensile stress under the specified loading of 550 psi tension, the load corresponding to this stress was applied to the first slab in one million repetitions with the lower limit of the loading corresponding to 50 psi compression. During this loading, extending over approximately one week, the cracks opened and closed one million times, with four repetitions per second, except at special rest periods. In spite of this rather severe test, the cracks became invisible and there was no noticeable permanent set.

The second slab was tested in a similar way, but three million repetitions were applied. Load-deflection diagrams for this test are given in Fig. 7 in which the loads are plotted in metric tons. The deflection curve *A* relates to the first static test when the load was increased until a number of cracks had developed. The load-deflection diagram of line *A* shows a curved portion commencing at a relatively low tensile stress similar to the curve illustrated in Fig. 4. In this case cracks occurred at a tensile stress of 900 psi. This relatively low stress may be explained by the fact that the prestressed beams for these two slabs were cast with a particularly dry mix in the bottom and there were a great number of cavities in the tensile zone, some of which were deep enough to allow a penknife to penetrate about 2 in. After the first million repetitions of loading, these cavities were thoroughly filled with high-alumina cement mortar. The mortar was allowed to harden one day before the static failure test of the first slab and the second million repetitions on the second slab. This filling proved satisfactory since both tests showed an excellent performance of the tensile zone of the composite member up to failure.

Curve *B* in Fig. 7 relates to the first static loading after completion of the first million repetitions, the latter being carried out within the same limits as the fatigue loading of the first slab. At the second million repetitions the upper limit of the loading was increased and corresponded to a nominal concrete tensile stress approaching 800 psi, which would occur at dead load plus 1.4 times live load. Deflection curve *C* relates to the first static loading of the third million of repetitions when the upper limit of load was further increased to correspond to a nominal tensile stress of approximately 950 psi, or dead load plus 1.6 times live load. In this case, it was necessary to raise the lower limit so as to reduce the amplitude to the maximum obtainable with the pulsator used. However, this reduced the range of loading.

Just before completion of the third million of repetitive loads, two tensioned wires fractured in gaps provided for fixing strain gages. Obviously, the fatigue strength of the steel was reached within these gaps where elongation

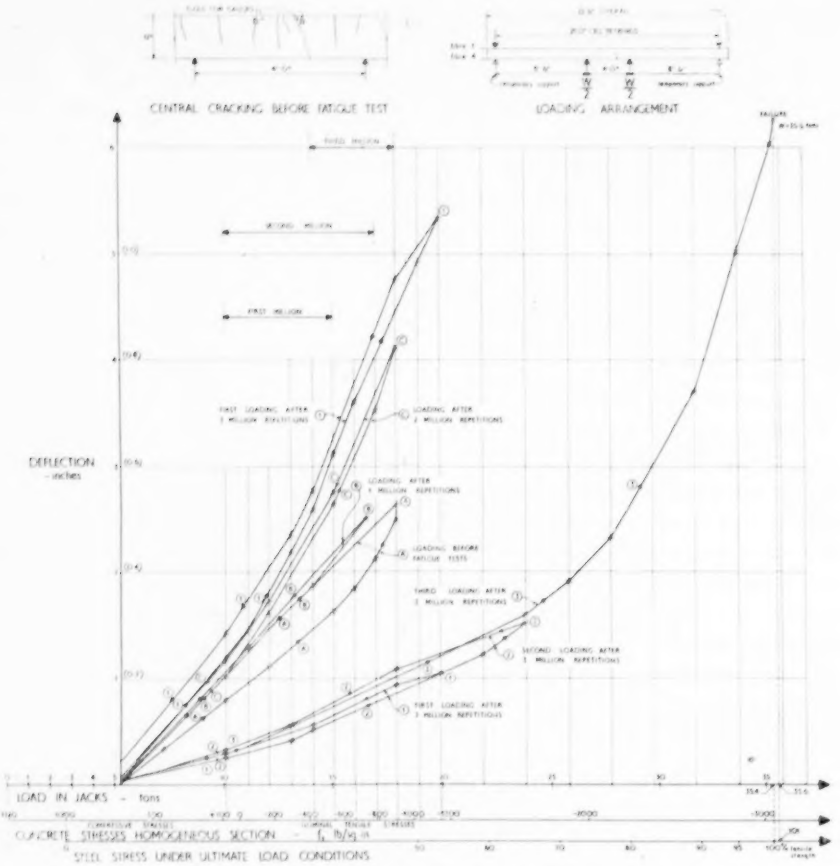

CENTRAL CRACKING BEFORE FATIGUE TEST
LOADING ARRANGEMENT
DEFLECTION - inches
LOADING BEFORE FATIGUE TESTS
LOAD IN JACKS - tons
CONCRETE STRESSES HOMOGENEOUS SECTION - f_c lb/sq. in.
STEEL STRESS UNDER ULTIMATE LOAD CONDITIONS

Fig. 7—Load-deflection diagram of slab No. 2 for fatigue test

was possible, and the concrete, though well bonded to the wire, did not restrain its elongation as was apparently the case with the wires within the slab. Immediately after the completion of the third million repetitions, the noticeable set due to deflection was relatively small, but the cracks did not become entirely invisible as had been the case after the completion of the first and second million repetitions.

The load-deflection curve 1 was taken after the completion of three million loadings and is plotted in Fig. 7 on two different scales of deflection. A comparison of load-deflection curves *A*, *B*, *C*, and 1 illustrates the behavior of prestressed concrete at different loading stages and indicates the influence of repeated as well as increased loading. Fig. 7 contains also a load-deflection curve leading up to failure. It will be noticed that the deflection was large and the failure load corresponds to a calculated average tensile stress in the remaining 58 tensioned and nontensioned wires which slightly exceeds the

calculated value based on the tensile strength of the steel. This stress was computed, as explained in connection with Fig. 4, on the assumption that both tensioned and nontensioned wires fully cooperate and balance the compressive force obtained for a rectangular stress block with a maximum stress equaling the prism strength of the concrete of approximately 6150 psi. With good bond, and consequently greatly limited width of cracks, it often occurs with under-reinforced sections that the calculated maximum steel stress exceeds the ultimate strength of the steel. This may be the effect of smaller contraction of the steel. Within the short width of a crack the actual sectional area of the steel may be greater due to less contraction, and thus the stress be smaller than in a test specimen of normal gage length for which the stress-strain curve of the steel is normally obtained. This strange behavior could be compared with that occurring in a notch of a tensioned steel rod; if the notch is relatively small, higher nominal stresses occur than the ultimate strength of the steel, the cause being that the actual contraction in the notch is less than that in an ordinary steel bar under the same stress.

In these fatigue tests the load was determined by gage readings at the jacks. However, it was important to ensure that the desired load acted on the slab. This obviously was influenced by the frequency of the slab which changed with the extent of cracking. This is a problem which requires special study, but it was outside the scope of this test which was to find out whether cracks closed completely after repeated loading and whether such repeated loading in a cracked state affects the ultimate resistance in a subsequent static loading. This was to be ascertained with the minimum of expense; consequently the endeavors were limited in these first two tests to whether the actual load applied to the slabs was in agreement with the load measured at the jacks. To obtain this, the deflection under static loading and the amplitude under dynamic loading were repeatedly compared. Thus, it was found in the first test that the amplitude under dynamic loading was considerably larger than the difference in deflection between the two limits of the range of loading in the static test. It can, therefore, be concluded that the actual loading was more severe than the nominal loading determined at the jacks. The amplitude was approximately 1.15 times the static deflection.

With the second test, the amplitude due to dynamic loading was only 85 percent, approximately, of the deflection due to static loading during the first one million repetitions, while the two values agreed well for the second million repetitions. However, with the third million, the amplitude due to dynamic loading was 35 percent greater than the static deflection within this range. Thus, it can be concluded that the first million repetitions was within a range of loading less than that measured at the jack, while the second million loadings were approximately in accordance with the assumption. However, the third million must have been considerably in excess of the assumed range of loading, since the frequent comparisons between static deflections and dynamic amplitude showed a difference as great as 35 percent. Obviously, from this it cannot be concluded between which

specific load the actual fatigue test was carried out, but it can be assumed that the range was considerably increased, most likely with a reduced lower limit and increased upper limit.

Fig. 8 shows the sets measured during the fatigue tests every morning before the pulsating loading commenced. The loading was continued for 12 hr. Temperatures were not taken when the sets were measured, but the measurements were taken each morning at approximately the same time before any sunshine was apparent which could have influenced the set. These measurements commenced only at the later part of the first million loadings. The ordinates in Fig. 8 represent the average distance of the slab at the center of the span related to a plane above the top level and were obtained as the difference of levels taken at the supports and the center. It is seen that the set gradually increased with an increase in the number of repetitions, but considerable recovery took place during the weekend when the specimen was allowed to rest. Obviously, the daily measurements give only the comparison of the state of test at the beginning of loading. In fact, after completion of the fatigue loading the set must have been much greater than on the following morning when some recovery probably had taken place, as indicated in the graph for the third million loadings. (The ordinates in Fig. 8 represent the camber due to prestressing which is reduced by loading.)

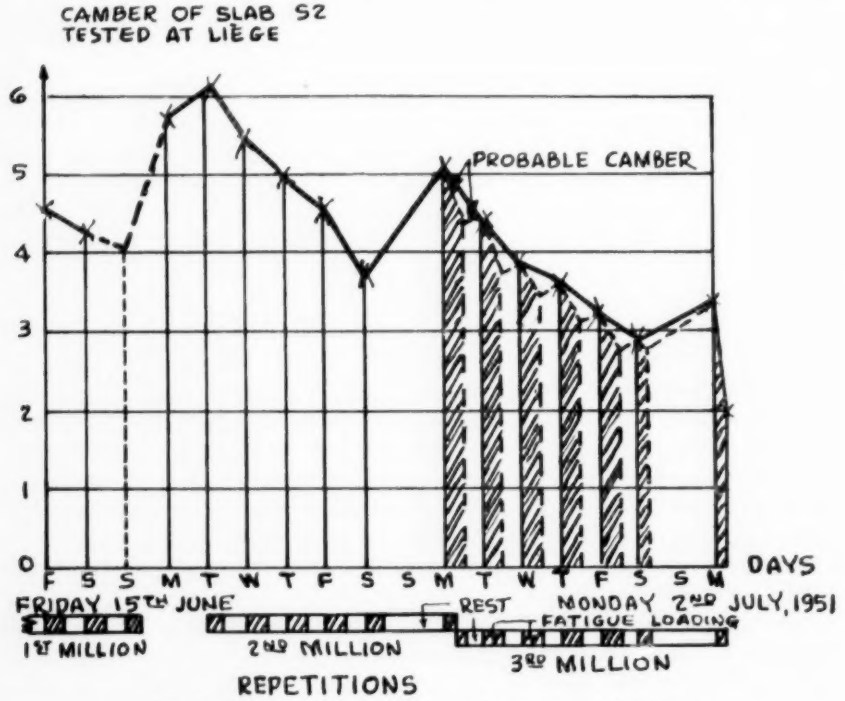
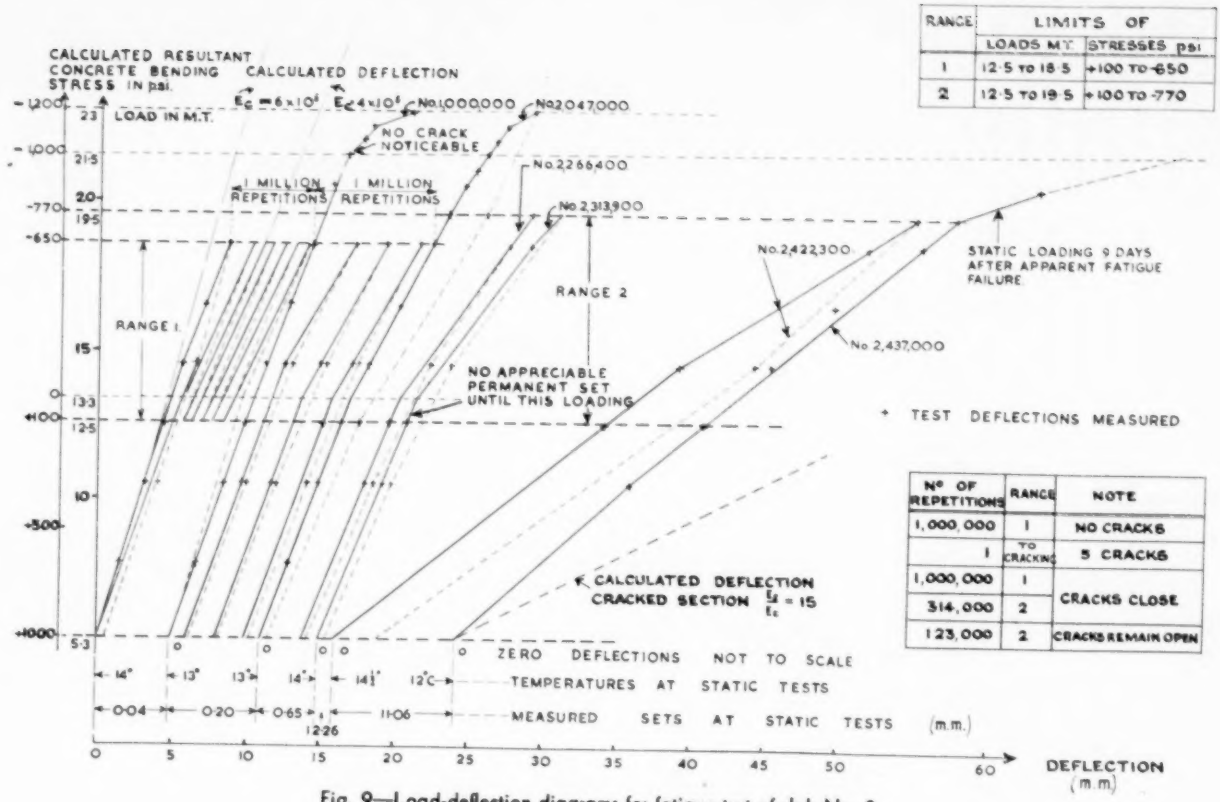


Fig. 8—Cracks and recoveries during fatigue test of slab No. 2

The third slab was tested in a different way, as seen from Fig. 9 which shows load-deflection diagrams. The first million loadings took place within a range corresponding to a compressive stress of 100 psi and a bending tensile stress of 650 psi. The inclination of the load-deflection curve gradually changed from a line corresponding to the value of E_c = 6,000,000 psi to one corresponding to E_c = 4,000,000 psi. During the first million loadings no cracks developed and the permanent set amounted to 0.04 mm (0.0016 in.). After completion of one million repetitions, a static loading was carried out, and at a loading corresponding to a bending tensile stress of 1000 psi and no cracks were noticeable. They became visible only at a load corresponding to a stress of 1200 psi, but it can be assumed that cracks may have developed at a lower load where a kink in the load-deflection diagram occurs. The second million repetitions took place in a cracked state within the same range as the first million as indicated in the graph. Permanent set after completion of the second million repetitions was 0.24 mm (0.01 in.). At loading No. 2,047,000 all cracks closed up entirely. Again a static load was applied up to the same maximum as after the first million loadings with little difference (Fig. 9). The third million loadings were commenced within a range corresponding to a larger upper load, as indicated in Fig. 9. When 266,900 repetitions had been finished, *i.e.*, after loading No. 2,313,900, the cracks still closed completely and no appreciable set occurred; the additional set measured at the 313,900th loading, before any recovery took place, was 0.65 mm (0.025 in.). Thus, the entire set was 0.89 mm or 0.035 in. However, on further continuing the fatigue loading, one side of the slab was strained gradually to a greater extent than the other and the cracks did not close completely but became wider and wider. Apparently bond and friction between wire and concrete, which was originally interrupted only at the cracks themselves, was gradually destroyed in their neighborhood, and the steel failed by fatigue similarly to the two wires in test slab No. 2. Gradual increase in permanent deformation took place and distinct reports were heard when individual wires broke. It was still possible to continue with the loading in the third range and to apply approximately 123,000 further repetitions between the loading when the cracks closed completely (loading No. 2,313,900), and the state when a great number of wires had fractured and permanent deformation with wide cracks occurred. One could conclude that fatigue failure had already taken place at loading No. 2,437,000. Fig. 10 illustrates the deformation of the slab after apparent fatigue failure, while Fig. 11 shows a crack which is wide at one side of the slab and relatively narrow at the other. This indicates that the loading was apparently not uniform and one side of the slab strained to a much greater extent than the other side.

To obtain more definite data about the loads applied to the third test slab (see Fig. 9), special strain and deflection measurements were taken during fatigue loading at frequent intervals and compared with corresponding measurements obtained from static loading, with a view to regulating the load at the jacks in the event of these strains and deflections differing between



RANGE	LIMITS OF	
	LOADS M.T.	STRESSES psi
1	12·5 TO 18·5	+100 TO -650
2	12·5 TO 19·5	+100 TO -770

Nº OF REPETITIONS	RANGE	NOTE
1,000,000	1	NO CRACKS
1	TO CRACKING	5 CRACKS
1,000,000	1	CRACKS CLOSE
314,000	2	
123,000	2	CRACKS REMAIN OPEN

Fig. 9—Load-deflection diagrams for fatigue test of slab No. 3

each other by more than 10 percent.

In Fig. 9 the load-deflection diagram No. 2,422,300 is also plotted, *i.e.*, a loading before apparent fatigue failure took place, and when the permanent set had further increased by 11.37 mm (0.448 in.) to a total of 12.26 mm (0.484 in.). Only those measurements are plotted where the temperature did not differ too much.

Fig. 10—Deflection of slab No. 3 after apparent fatigue failure and 9 days recovery before static failure test

Nine days after apparent fatigue failure a static loading test was carried out, the deflection diagram of which is plotted in Fig. 9. At the beginning of the static test, the increase in set, as compared with the state of loading No. 2,422,300, amounted to 11.06 mm (0.436 in.) and the entire set was 23.32 mm (0.918 in.). During rest a considerable recovery must have taken place and the load-deflection line is much steeper than at apparent fatigue failure; the maximum load reached exceeded the upper limit of the second range by 11 percent.

When comparing the test results of the third slab with those of the other two slabs, it must be pointed out that the conditions were apparently quite different, mainly due to the fact that in the latter case the loading during the third million repetitions was not uniformly distributed over both sides of the slab; one side was loaded to a much greater extent than the other, causing an earlier destruction of bond adjacent to the cracks. When considering the third range of the second slab, it must be taken into account that the range was reduced since the lower limit had to be raised although the upper limit was remarkably high. However, even the second million repetitions of the second slab, which corresponds approximately to the second range of the third slab, was completed without any appreciable permanent set. Early fatigue failure occurred in the third slab due to overstressing at one part of the slab.

Fig. 11—Main crack of varying width in slab No. 3 after apparent fatigue failure

This gave a welcome opportunity to observe fatigue failure of a prestressed concrete structure.

It often is usual to ascertain the endurance limit (the stress corresponding to the upper limit of a loading for which no failure occurs) after one million or two million repetitions applied from zero to the upper limit. However, this is of little practical use in prestressed concrete since the range should be chosen between actual limits, the lower limit being the dead load and the upper limit corresponding to maximum possible loading of the fatigue condition, ensuring a sufficient factor of safety against failure. The foregoing descriptions have shown some interesting features of these fatigue tests, but there are many strain measurements and data not yet evaluated which should give further valuable information.

ROUTINE TESTS

Routine acceptance tests have been made by the Civil Engineer's Department, Eastern Region, British Railways, since early 1949 to check the quality of prestressed concrete. These tests represented the manufacture of nearly 3000 units between 12 and 102 ft long. One beam of each batch manufactured in the same pre-tensioned line was selected and subjected to a test loading corresponding to a tensile stress of 750 to 800 psi, the rate of testing being related to the actual effective prestress. Similar tests were made on individual large beams containing post-tensioning cables, the test load corresponding to a tensile stress of 650 psi. At such tests no cracks were supposed to occur, and the deflection should not be greater than the value corresponding to E_c = 3,750,000 psi. In many cases the deflection corresponded to E values between 5,000,000 and 6,500,000 psi.

In numerous similar tests between 1949 and 1952 it was not necessary to reject a single beam. In one instance the loading was sustained more than 30 days with greatly increased deflection, but no cracks developed. The losses taken into account are based on strain measurements extending over a year, and the various tests have indicated that the calculated values agree well with the actual tests.

Since 1952, in a few cases in different concrete plants, hair cracks developed under the required acceptance test loading. There was always a special cause for this. In one case, it was established that the cracks must have developed before the prestress was transferred to the concrete, thus the cracks opened as soon as tensile stresses occurred and became visible at a loading corresponding to approximately 300 psi tensile stress. Apparently shrinkage cracks had occurred due to lack of moistening between steam curing and the time when the prestress was transferred to the concrete. No further trouble has occurred since a sprinkler arrangement was provided for curing the products. In another case early cracking was caused by the tensioning stress applied being less than the stress required due to incorrect determination of the required total elongation of the steel.

These acceptance tests prove that strict supervision is essential, and that

it is possible to discover any faults in manufacturing. Increased factors of safety against cracking and failure are often suggested since it is impossible to rely on correct manufacture. However, the author is of the opinion that an increased factor of safety does not prevent the occurrence of faults more serious than one would expect. For instance, the use of a wire with unsatisfactory surface conditions may cause slipping, particularly if in addition the concrete does not attain the required strength. Thus it is possible to expect a high factor of safety against cracking on the basis of the design but the construction may crack even under the working load. It is, therefore, essential to obtain complete agreement between design and manufacture. If such agreement is ensured, there is no need to increase the factor of safety be it that against cracking or failure. If, for instance, freedom from cracks under a loading corresponding to a stress of 750 psi is ensured, and this even for a sustained loading of 30 days, one can be sure that no cracks will develop under a permanently sustained loading corresponding to 600 psi. After the third loading test, the permissible tensile stress for high road bridges was increased from the 550 psi previously mentioned to 650 psi.

CONCLUSIONS

Conclusions 1 and 2 are based on extensive routine acceptance tests of concrete made at different plants with regard to pre-tensioning, and to precast members with post-tensioning cables which are well grouted, due allowance being made for the losses. Conclusions 3 to 10 are based on the three fatigue tests described. These important conclusions are obviously limited to concrete of a 28-day cylinder or prism strength of approximately 6000 psi, and to a modulus of rupture of 1000 psi, ascertained with well-bonded, pre-tensioned and nontensioned wire. This feature is of cardinal importance, and it would appear that the actual results are to a certain extent due to the cooperation of nontensioned wires, which seem to take up a great part of tensile stress just before the fatigue resistance of the tensioned wires is exhausted.*

1. No cracks ought to occur in an acceptance test in any beam with well distributed pre-tensioned wires under a loading which corresponds to a concrete bending tensile stress of 750 to 800 psi, even if the load is sustained for some time.

2. No cracks ought to occur in an acceptance test in a prestressed concrete beam with post-tensioned cables which are well grouted under a loading which corresponds to a tensile bending stress of 650 psi.

3. No cracks should occur in a prestressed construction with pre-tensioned wires under a fatigue loading, the upper limit of which corresponds to a tension bending stress of 650 psi.†

4. A fatigue loading according to conclusion 3 does not affect adversely the commencement of cracking at the subsequent static loading which corresponds to a

*Fatigue tests in 1954 have also indicated that a great fatigue resistance can be ensured without additional nontensioned wires. In this case, nearly nine million repetitions were applied to a beam in a cracked state. The upper limit of the range was gradually increased from ½ to ⅔ of the static failure load with a lower limit of ¼ the static failure load.

†In view of the test results of the 1954 tests, this has to be modified by the proviso that the entire range of fatigue loading is not appreciably higher than 650 psi. With a range of 910 psi and a maximum tensile stress of 700 psi, cracks developed under a fatigue loading, while freedom from cracks was obtained with a range of stresses of 610 psi and the same maximum tensile stress of 700 psi after one million repetitions.

modulus of rupture of 1000–1100 psi.

5. When cracks have occurred in a prestressed structure and a fatigue loading takes place with an upper limit corresponding to a nominal tensile stress of 650 psi, the cracks become invisible on removal of the load and no noticeable deformation remains, even after the application of a million loadings. This limit may be increased even further.

6. Several million repetitions of loadings up to a limit not exceeding the load mentioned in conclusion 5 do not affect the subsequent static failure load, which is the same as if no previous fatigue loading had taken place.

7. Even if the cooperation between wires and concrete is gradually destroyed and the approaching failure is clearly noticeable by large permanent deflection and wide cracks, the structure is capable of withstanding numerous loadings without failure (*e.g.*, 124,000), although one wire after the other may fracture. Obviously, this occurs only at a much higher loading than that mentioned in conclusion 5.

8. Even after apparent fatigue failure, a prestressed structure of the type in question may be capable of withstanding a still higher static failure load than that corresponding to apparent fatigue failure.

9. Full cooperation of tensioned and nontensioned wires of 0.2-in. diameter and a strength of 224,000 psi and a concrete of 28-day cylinder strength of approximately 6000 psi has been ascertained in static loadings up to failure (even after previous fatigue loading). Stress of the maximum tensile force corresponded to the maximum tensile strength of the steel. (The nontensioned wires were placed in twin-twisted pairs to increase mechanical bond.)

10. Where the wire is not embedded in the concrete the cooperation is interrupted and conditions quite different from those with bonded wire apply, causing an early fracture of the wire. Consequently, in constructions in which the wires are not bonded, a much lower stress ought to be considered for the steel in the ultimate strength, *i.e.*, only the fatigue strength of the wire, which is approximately two-thirds of the static strength.

It must be added that all the conclusions relating to fatigue tests are based on a loading at which repetitions were applied continuously over many hours without rest. In practical use the conditions are quite different, except perhaps for some railroad bridges and special constructions, *e.g.*, turbine foundations. Thus the loading conditions were much more severe than under ordinary conditions.

ACKNOWLEDGMENTS

The author expresses his thanks to J. I. Campbell, civil engineer, Eastern Region, British Railways, for permission to use the research data mentioned; thanks are also due to Mr. Uyttendale of Sealithor Developments Co., Ltd., for the permission to show particulars not yet published of the third fatigue test on the third slab.

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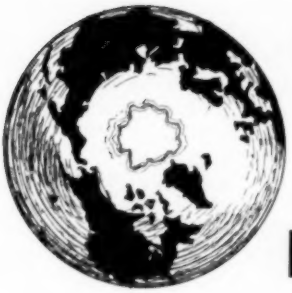

Letters from Readers*

BY WAY OF SYNOPSIS

William R. Lorman discusses Norwegian Calcite as a concrete admixture.

Irving B. Rau discusses the use of electrical resistance strain gages in concrete research.

Gerald Gillan discusses test methods described in the ACI Journal paper "Failure of Concrete Under Combined Tensile and Compressive Stresses."

A. F. Pillsbury discusses the ACI Journal paper "Control of Surging in Concrete Pipe Distribution Systems."

Kuang-Han Chu and David P. Billington discuss design equations in the ACI Journal paper "Economical Design of Prestressed Concrete Beams."

E. Walter and Rudolph C. Valore, Jr., supplement the ACI Journal paper "Cellular Concretes" with information on another process for making cellular concrete.

Norwegian Calcite as a Concrete Admixture (LR 51-9)†

In a letter in the May 1954 ACI Journal,‡ Dr. Eivind Hognestad summarized the technical information developed in recent years concerning "Activite" (also known as "Aktivitt"). Dr. Hognestad pointed out that those interested in calcite as a concrete admixture "may wisely study the extensive test data of Norwegian and Danish laboratories and draw his own conclusions." He also indicated that the controversial issues regarding this admixture are yet to be resolved.

In February, 1951, the writer had occasion to review a number of assertions made in connection with the Norwegian calcite commercially known as Activite. The only information available at the time was that furnished by Nicolas Bierqué, Casablanca, Morocco, and consisted of diverse statements, excerpts from Norwegian journals, and various test data, all of which apparently were prepared or provided by the following: Arne Daniels, geologist, Smestad, Oslo; Tom F. W. Barth and Christoffer Oftedahl, both of the Institute of Mineralogy, University of Oslo, Blindern, Oslo; the Oslo Institute for Testing Materials; and Carsten Friis-Baastad, Lakeville, Conn. Soon thereafter the writer conducted a literature survey (extending back to 1929) with the hope of finding substantiative data but unfortunately no such germane evidence was revealed.

To obtain worthwhile laboratory test data,

*A part of copyrighted Journal of the American Concrete Institute, V. 26, No. 4, Dec. 1954, *Proceedings* V. 51. Separate prints of the entire Letters from Readers section are available at 35 cents each. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†The opinions or statements in this paper are the private ones of the writer, and are not to be interpreted as an expression of official U. S. Navy policy nor are they necessarily a reflection of the official views of the U. S. Naval Civil Engineering Research and Evaluation Laboratory.

‡"Calcite (Aktivitt) as Concrete Admixture (LR 50-17)," ACI Journal, May 1954, *Proc* V. 50, p. 801.

a tremendous number of test specimens would be necessary and observations thereon would extend over several years. An experimental program of this nature most likely would develop into a vast project dealing with comparative characteristics of concrete admixtures in general; properly, it would include pozzolanic materials, a subject which by itself is of exceedingly great scope. It is believed that if such a testing program were undertaken it should be truly comprehensive to establish all-inclusive proof concerning the attributes of Activite. Rather extensive testing facilities and certainly ample financial support would be necessary for sustaining a long-term testing program of the type indicated later in this paper.

Study of the assertions made for Activite

The subparagraphs appearing below deal with the various characteristics of Activite when used as a concrete admixture. Although each point concerning Activite received careful consideration and was studied with reference to established or commonly-accepted principles, it is obvious that only extensive laboratory experimentation will serve to prove or disprove the various points. It is apropos at this time to point out that recent experimentation by Inge Lyse* has revealed that: (a) calcite concrete does not withstand effects of freezing-thawing action as well as conventional concrete; and (b) concrete incorporating 25 percent calcite as a portland cement substitute demonstrates considerably lower compressive strength than conventional concrete.

1. Activite supposedly is pure calcite and is obtained from Norway. It is maintained that this material reacts with the free lime in portland cement concrete and transforms that lime into a mineral substance which endows the concrete with unusual durability. The optimum effect of Activite as an admixture supposedly is had when used in amounts of about 25 percent (by weight) of the portland cement in the mix.

It is known that loss of lime results in strength decrease; when about 50 percent of the original lime has been removed from average concrete, loss of strength is inevitable. If calcium carbonate is removed from the concrete surface by the dissolving action of surface water, calcium hydroxide from the interior will diffuse to the surface and will be deposited as $CaCO_3$ by virtue of the CO_2 or carbonates present in the water. When $Ca(OH)_2$ is thus transformed the interstitial water within the concrete becomes less saturated and additional $Ca(OH)_2$ goes into solution. As a consequence, there exists continuous movement of $Ca(OH)_2$ from the interior to the surface of the concrete; the end result is loss of lime throughout the entire mass of concrete. However, this conversion of $Ca(OH)_2$ to $CaCO_3$ is extremely slow.

Carbonation, although a common process where concrete is exposed to the elements, generally occurs in the relatively thin portion comprising the exterior surface; when found to excess in connection with fractures in the concrete, carbonation is an indication of deterioration.

As far as known to the writer, Activite is pure calcite or $CaCO_3$. It is inconceivable, therefore, that $CaCO_3$ will react with CaO or $Ca(OH)_2$ and as a result impart high durability to the concrete.

The $CaCO_3$ found in concrete generally is in the form of calcite which is innocuous as far as alkali-reactivity is concerned. In this connection, the presence of Activite in a concrete mix containing high-alkali cement and reactive aggregate would have no logical basis from the viewpoint of imparting durability to the concrete.

2. It is alleged that Activite will react with humus and eliminate the effects of this extraneous material in a concrete mix. The reaction supposedly involves calcite neutralizing humus with the result that adhesion of cement paste to concrete aggregate particles coated with humus will be possible; it is claimed that adhesion is accomplished by virtue of the adsorptive binding of colloidal humus to the Activite particles. This is not a logical explanation.

As humus is the organic portion of soil and is formed by the partial decomposition of vegetable or animal matter, it appears unlikely that $CaCO_3$ would react with organic compounds usually found in humus. Moreover, the presence of organic impurities beyond the allowable limits (as defined by

*"Discover New-Type Cement, Norwegians Say (LR 46-46)," ACI JOURNAL, Jan. 1950, *Proc* V. 46, p. 392.

ASTM) would be considered objectionable insofar as good concrete ingredients are concerned.

3. It is asserted that Activite improves plasticity and results in lower mixing water requirements. If the fineness of Activite is wholly minus No. 200 (U. S. standard screen) it is possible that plasticity of the concrete may be improved. However, it should be noted that the introduction of finely-powdered materials in concrete mixes not deficient in fines generally necessitates additional mixing water because of the absorptive character of the admixture. The presence of additional water will obviate any benefits that otherwise would accrue from the presence of the admixture material (for example, increased drying shrinkage).

If Activite is water-repellent, if it possesses the same volume wet or dry, if it is chemically inert, and if it serves as a lubricant (as, for example, an air-entraining agent does), then and only then would its use appear justifiable in regard to improved plasticity of the concrete and lesser mixing water requirements. However, $CaCO_3$ is not water-resistant and may be altered by water containing CO_2 in which case the H_2CO_3 reacts with $CaCO_3$ to produce soluble $Ca(HCO_3)_2$.

4. It is contended that Activite will result in homogeneous concrete by virtue of its ability to counteract segregation. It is well known that segregation of the aggregate (notably coarse) from the mortar occurs when the concrete consistency is too wet. Unless Activite's properties are such as meet the requirements for an improver of plasticity (outlined above in 3), it is questionable that Activite eliminates segregation in portland cement concrete.

5. Activite supposedly increases the compressive and flexural strength of concrete. Although the technical information made available through Mr. Bierqué contained some graphs and tables to support this thesis, the experimental data shown were rather meager and consequently not conclusive. For example, no data were shown relative to the types or quality of the various aggregates employed in the concrete and mortar tests cited. Moreover, the quality of the test data was questionable, there being no indication as to quantities and sizes of specimens nor to curing and storage conditions. In the mortar tests that portion of sand finer than No. 50 had been replaced by Activite; whether or not such was the case for Activite concrete was not shown. In some tests Activite was used as a partial substitute for sand, in other cases as an addition to the aggregates in the concrete mix. In actual construction operations, the cost of removing part of the sand and replacing with Activite would be prohibitive.

It is known that finely-powdered compounds, when used as filler material in a concrete mix, usually result in increased density and higher compressive strength; but such beneficial effects of void-filling generally apply to lean or harsh mixes. In the absence of specific data concerning unit weights, specific gravities, and other pertinent characteristics of the aggregates used in the tests, it is impossible to conclude that increased strengths most likely occurred because the Activite served as a void-filler.

Despite these explanations of possible beneficial effects of Activite, there exists a belief that calcite concrete is inferior to ordinary concrete in regard to compressive strength, as has been noted above.

6. Activite concrete supposedly is durable in sea water, in acidic fresh water, and in relatively strong acid solutions such as H_2SO_4 or HCl. It is asserted that sea water has little ability to transform the "amorphous Activite concrete" whereas with ordinary concrete sea water reacts with the calcium silicates and aluminates, all accompanied by displacement of calcium by magnesium. Only the surface of Activite concrete is attacked by acid solutions, it is pointed out, but with the reservation that H_2SO_4 results in the formation of gypsum (which by definition is $CaSO_4 + 2H_2O$) in the concrete pores and capillaries thus precluding further entry of acid solutions into the concrete mass.

These assertions appear to be questionable. It has been established, in the literature dealing with concrete, that the strength of concrete at any stage of attack by acid waters is dependent upon the ratio of mixing water to amount of lime remaining in the concrete. Lime leached from concrete is derived partly from crystalline $Ca(OH)_2$ and partly from the cement gel. These processes are complementary rather than competitive and contribute to loss of concrete strength regardless

of whether the lime is redeposited as $CaCO_3$ elsewhere within the mass or completely removed from it (see 1 above).

The main cause of disintegration by sea water is a chemical reaction between the hardened cement paste and the magnesium salts in the sea water. This chemical attack consists of two principal reactions: (a) sea water sulfates and C_3A combine to form compounds that expand by absorbing water of crystallization, and (b) leaching of lime either by replacement with magnesium or by formation of the more soluble $Ca(HCO_3)_2$. The formation of gypsum is accompanied by an increase in volume which action tends to crack the already hardened concrete; and the production of sulfo-aluminates causes swelling which is even more destructive. The gypsum created in this transformation in all likelihood is washed away by wave action; abrasion as well as freezing-thawing action also hasten the disintegration. It would seem that the transformation of $Ca(OH)_2$ into $CaCO_3$, as outlined in 1 above, would greatly increase resistance to sea water but unfortunately the absorption of CO_2 is too slow to be practicable in this respect.

In view of the above, it follows that no admixture affords complete protection against the destructive action of sea water and acidic waters.

7. Activite concrete supposedly is highly impermeable. It is known that finely subdivided compounds used as fillers generally result in reduced permeability. Therefore, it is possible that Activite concrete, by virtue of its better-filled pores, may tend toward improved impermeability. But no permeability test data were found (in the technical information available) to support this contention.

8. It is alleged that Activite concrete has high abrasion resistance. It would seem that the quality of high resistance to surface wear is not related whatsoever to the presence or absence of a comparatively inert mineral admixture such as calcite. Good abrasion resistance of concrete depends upon aggregate particle hardness and toughness and basically is a function of properly-hydrated cement particles. By comparison, the Moh hardness of calcite is 3, whereas a good quartz is 7.

9. For mass concrete purposes, the statement was advanced that Activite will insure lowest minimum cement content and good workability. No specific data were presented and therefore it was not known exactly what "lowest minimum" cement content amounted to. However, it is not uncommon to design mass concrete mixes (6-in. maximum aggregate) on the basis of about 4 sacks of cement per cu yd. It is not clear how the presence of Activite would warrant any lesser cement content unless the Activite is used as a partial replacement of cement. Pozzolans of the calcined opaline type have been used as replacement of portions of the cement, and for lean mixes of mass concrete perhaps could satisfactorily replace as much as 50 percent of the portland cement. As Activite is not a pozzolan, and as there is no evidence to indicate that its properties simulate those of a pozzolan, it is doubtful that any appreciable reduction in cement content will be effected by introduction of calcite in the concrete mix.

Conclusions

In general, the use of an admixture is warranted only if the standard ingredients do not economically produce satisfactory concrete. No admixture will remedy defects due to poor cement, excessive water, poor aggregate, improper aggregate gradation, and similar discrepancies. To determine whether or not Activite imparts some special quality to portland cement concrete, the necessity for extensive laboratory investigation is evident. Judging the value of an admixture involves considerably more criteria than that of concrete strength alone. It is believed essential, therefore, that the effects of Activite upon the following characteristics of concrete are basic to any definite decision as to the relative merits of the admixture:

Economy—(1) Cement content and (2) water-cement ratio.

Workability—(1) Consistency, (2) segregation, and (3) bleeding.

Durability — (1) Weathering resistance (freezing-thawing, absorption), (2) chemical attack resistance (sulfates, alkali-reactivity), (3) abrasion resistance, (4) impermeability, (5) volume change (thermal expansion, drying shrinkage, growth, creep), and (6) bond to reinforcement.

Elastic properties—(1) Modulus of elasticity, (2) Poisson's ratio, (3) compressive strength, and (4) flexural strength.

It is hoped that the foregoing discourse will serve as a suitable supplement to the presentation by Dr. Hognestad and to information which appeared previously in the ACI JOURNAL. A consolidation of all these views may be of assistance to those who contemplate investigating Activite (Aktivitt) as an admixture for portland cement concrete.

WILLIAM R. LORMAN, Materials Engineer, Structures Research Department, U. S. Naval Civil Engineering Research and Evaluation Laboratory, Port Hueneme, Calif.

Lightweight Prestressed Concrete (LR 51-10)

I read with interest Fred E. Koebel's article in the March 1954 JOURNAL.* Attention is directed to the number of strain gages used and their location on the beam.

The writer has had one experience that was anything but good using type A-12, SR-4 strain gages in a circuit using a galvanometer and an Anderson-Fluke strain meter to measure the strains in a prestressed concrete beam.† In this investigation five strain gages were mounted on each side of a rectangular prestressed beam at approximately the sixth-points. The gages were attached to a smooth concrete surface that had been rubbed with an emery cloth, soaked with acetone, covered with Duco cement, a light application of acetone was applied, a final application of Duco cement was applied, then the strain gages were applied to this surface. This was the procedure used in the MIT laboratories at that time. Mr. Koebel attached the gages to an aluminum foil which was in turn attached to the concrete.

The writer's ten gages were located at the mid-length of the beam and third-point loading was applied by a testing machine. From these ten measured strains an average straight line was finally determined, not, however, as easy as might be expected. The writer is of the opinion that a percentage of error as great as 10 percent could be introduced depending on how the strains were plotted and on how a straight line was drawn in.

The writer notes that in Mr. Koebel's investigations that gages were applied at the outer fibers and some averaging was done. It is suggested that future investigators use additional strain gages at various points on the sides of the beam to help them when averaging to decrease the chance for error from this source. The writer does believe that measuring of concrete strains by electrical resistance wire strain gages is in itself not too accurate a procedure.

IRVING B. RAU, New Orleans, La.

Failure of Concrete Under Combined Tensile and Compressive Stresses (LR 51-11)

G. M. Smith in the October 1953 JOURNAL‡ has used an interesting piece of testing equipment and is to be complimented for his ingenuity. He states that "friction between bearing plates and specimen has little, if any, effect on failure." The writer believes that the effect of friction between the bearing plates and the specimen is significant enough to require consideration.

Denoting the bearing plate width, a; the specimen width, b; the specimen depth, d; and the coefficient of friction between steel and concrete, f; and assuming the line of fracture occurs at the middle of the bearing plate, one may analyze the problem as follows. The frictional force opposing the ordinary beam action is $\frac{1}{2}(faS_c)$ lb per in. of depth of specimen. The resultant force

*Koebel, Fred E., "Lightweight Prestressed Concrete," ACI JOURNAL, Mar. 1954, *Proc.* V. 50, p. 585.

†Rau, Irving B., "A Comparison of the Experimental and Theoretical Stresses in a Prestressed Concrete Beam," Master of Science Thesis, Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology, 1949.

‡Smith, G. M., "Failures of Concrete Under Combined Tensile and Compressive Stresses," ACI JOURNAL, Oct. 1953, *Proc.* V. 50, p. 137.

acting on the specimen in each of four quadrants of contact area between bearing plate and specimen is $\frac{1}{4}(fadS_c)$, the forces in the first and fourth quadrants forming a couple opposing that of the forces in the second and third quadrants. The moment of the couple is $\frac{1}{8}(fad^2 S_c)$. Because motion actually occurs at the interface, the assumption of uniform frictional force is considered justified.

The case studied by the author has values of $a = 1$ in., $b = 4$ in., and $d = 3$ in. Assuming, for the sake of a numerical example only, a value of $f = 0.24$, the moment due to friction is equal to $0.27\ S_c$ in.-lb. The added apparent flexural stress S_t is then $0.045\ S_c$ psi. This is the amount by which each point of Fig. 3 in the original paper, except those on the axes, should move to the left.

If the assumed coefficient of friction is valid, the author might now find a "remarkable correlation" with a stress ratio function

$$\left(\frac{S_c}{S_u}\right)^2 + \frac{S_t}{S_v} = 1$$

which is parabolic in contrast to his elliptical function. This function has an added attractive feature not possessed by the elliptical function, namely that its tangent at the point marked S_u in Fig. 3 is not horizontal. It is known that in the plus-plus domain (second quadrant of Fig. 3) the stress-ratio function rises from S_u and a discontinuity at this point seems improbable.

In the event that jack capacity does not limit him, the author could verify or reject the writer's ideas by using bearing plates 2 in. wide instead of those used. The results, if the writer is on the right track, should throw the uncorrected points farther to the right.

GERALD GILLAN, Professor of Civil Engineering, Pennsylvania State University, State College, Pa.

Control of Surging in Concrete Pipe Distribution Systems (LR 51-12)

Messrs. Hale, Glover, Terrell, and Simmons have ably and simply reported* on their extensive investigations of the phenomenon of surge in open type concrete pipe irrigation water distribution systems. Studies which Prof. E. H. Taylor and I have made,[1,2,3] on phases of this same subject substantially agree with their findings. However, the following comments appear pertinent.

Vents for surge control

Vents on our model completely eliminated the initiation of surge when correctly placed and of adequate size. By surge initiation we refer to what the authors call "auto-oscillation." We concur that vents have no significant effect upon the amplification of surge. As regards failure of vents to offer relief on the Coachella prototype, I have been unable to find a single lateral on that system where there is the necessary combination of correct placing, adequate size, and adequate number of vents to provide correction. I am well aware that I went far astray in my conclusions originally when these were based solely upon field observation. It is possible that the Bureau encountered a similar difficulty—locating and installing vents in the field before there was sufficient model experience to indicate proper design. It is to be hoped that vents can yet be given a fair field trial instead of dismissing them with over-emphasis of their obvious limitations.

Control of surging by use of covers

From scattered observations, it is my conclusion that some of the untrustworthiness ascribed to vents could be better ascribed to covers as a paliative for air entrainment.

Inherent instability of the open system

This paper brings out again that the open type pipe system is inherently unstable, and should be avoided in all future design. This system, with periodic overflow devices to control pressure and keep it low, has permitted economies by making possible the use of inexpensive concrete irrigation pipe (non-reinforced, tongue-and-groove mortared joints). For a variety of reasons, there is a tendency now to depend more on pressure

*Hale, C. S., Glover, R. E., Terrell, P. W., and Simmons, W. P., Jr., "Control of Surging in Concrete Pipe Distribution Systems," ACI JOURNAL, Mar. 1954, *Proc.* V. 50, p. 573.

pipe for district systems. Thus, there is little economy in not going to the more flexible, and cheaper to operate, closed system. Where irrigation pipe can be used, the semi-closed system (float valve controlled) provides desirable operating characteristics.

References

1. Taylor, E. H., and Pillsbury, A. F., "Preliminary Notes on the Phenomenon of Surge as Observed in Open Type Irrigation Pipe Distribution Systems," *Technical Progress Notes*, Department of Irrigation and Soils, University of California, Los Angeles, 1952.

2. Pillsbury, A. F., and Taylor, E. H., "Hydraulic Characteristics of Pipe Systems for Irrigation Enterprises," *Agricultural Engineer*, V. 34, 1953, pp. 837-840, 842.

3. Taylor, E. H., Pillsbury, A. F., Ellis, T. O., and Bekey, G. A., "Unsteady Flow in Open Type Pipe Irrigation Systems," *Proceedings*, ASCE, V. 79, Separate No. 369.

A. F. Pillsbury, Professor of Irrigation, Department of Irrigation and Soils, University of California, Los Angeles, Calif.

Economical Design of Prestressed Concrete Beams (LR 51-13)

David P. Billington has presented an interesting study on the economics of simple prestressed concrete beams in the September 1953 Journal.* However, his principal Eq. (6) or (6a) can be simplified into the following form:

$$\frac{\Delta_{te}}{bh} = \sqrt{\frac{F}{D_c b}}\,\frac{K_1}{L} - K$$

where $K_1 = 2\sqrt{1 - \frac{2}{3\alpha}}$, $K = 1/2\alpha$, and $\alpha = h_t/h$.

The values of K_1 and K can be found from the curves in Fig. 1. The above formula has the following advantages: (a) it is obviously dimensionally consistent; and (b) it requires fewer coefficients to be computed and picked from curves than the original formula.

Kuang-Han Chu, Structural Designer, D. B. Steinman, Consulting Engineer, New York, N. Y.

* * * * *

The author is grateful to Mr. Chu for his simplification of Eq. (6a). For a given set of dimensions and prestressing force this simplification reduces the operations involved in checking a section and is, therefore, of use to the designer.

There is one further remark that should be made on the paper. The formulas are based on a condition of zero tension in the bottom fibers under full load. A section designed according to the concepts developed in the paper may, particularly in long span girders, be overstressed in compression at the bottom fibers when the prestressing force is applied. Such overstressing can be avoided by prestressing in two stages, the addition of mild steel in the bottom flange, or an increase of concrete in the bottom flange. It is the author's opinion that the last solution is usually the least economical.

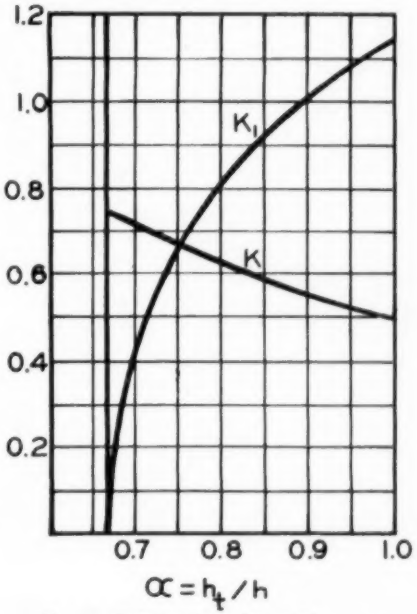

Fig. 1 — Relationship of k and k_1 to α

David P. Billington, Bloomfield, N. J.

*Billington, David P., "Economical Design of Prestressed Concrete Beams," ACI Journal, Sept. 1953, *Proc.* V. 50, p. 73.

Cellular Concretes (LR 51-14)

Rudolph C. Valore, Jr., has an interesting article about cellular concretes in the May 1954 JOURNAL.* However, in this history of cellular concretes he omits completely the accomplishments by the Porete Mfg. Co. The Porete Co. was the first concern in the United States to produce precast slabs made of cellular concrete, starting in 1920. The process was based on these patents: (1) U. S. Patent 1,313,931—Ernest Walter—Aug. 26, 1919; and (2) U. S. Patent 1,344,324—Ernest Walter—June 22, 1920.

These patents cover a process for mixing portland cement and sand with small globular pellets made of an extractable material insoluble in water, which melts at a temperature where it will not injure the concrete. The material used for this purpose was paradichlorbenzol and later a combination of paradichlor with napthalene. The mixture was poured into steel molds for making wire reinforced flat slabs. Most of the slabs were 24 in. wide, 32 in. long, and 1¼ to 2 in. thick. The next day, after the slabs had set, they were stripped and stacked in cars and introduced into an 8 ft diameter, 25 ft long steel tank in which they were exposed to superheated steam slightly above atmospheric pressure. A large part of the dichlorbenzol melted and was recovered at the bottom of the tank. The balance evaporated and formed a water and dichlorbenzol vapor mixture which was condensed and recovered. The slabs came out of the superheater bone dry and made an exceptionally strong lightweight concrete of about 70 to 75 lb per cu ft with large holes of about ⅛-in. diameter.

The melted dichlorbenzol was pumped into a flat dish which was provided with small nozzles which were submerged in hot water. Because dichlorbenzol is about 30 percent heavier than water, it formed small droplets which sank down into a cool zone of water where the droplets congealed and where a conveyor brought them back to the mixer. The loss of dichlorbenzol was negligible. The dichlorbenzol cost only 2 cents per lb in 1920.

This process was used for 5 years until 1925. The Porete Mfg. Co. then made a license arrangement with John Rice in California to use his foaming process, which was simpler and somewhat less costly.

E. WALTER, President, Porete Mfg. Co., North Arlington, N. J.

* * * *

Relative to Mr. Walter's process for making cellular concrete, my paper omitted mention of it. This "volatile aggregate" process might have been mentioned briefly in the first footnote on p. 782 where a number of other chemical processes used, apparently, on only a limited scale commercially, were mentioned.

Within the limitations of space and time my paper purported to review the present status of the field of cellular concretes. The history had to be dealt with selectively and only the processes that had been used for some time, and those that appeared most promising, were described in any detail.

The process Mr. Walter describes was apparently modified by the addition of aluminum powder to the mixture according to Mr. Walter's U. S. Patent 1,829,381, granted Oct. 27, 1931. The author possessed, at the time of preparing the paper on cellular concretes, a brochure titled "Porete-Fill," *Circular* No. 43, 1930, Porete Co. At the time that *Circular* No. 43 was prepared the Porete Co. had apparently discarded the volatile aggregate process for a preformed foaming process, as Mr. Walter has stated.

Mr. Walter's process might be grouped with a number of soluble or volatile aggregate types of concrete. In Mr. Walter's 1931 patent a volatile liquid, insoluble in water, was used. In an earlier process crushed ice or snow was incorporated into a mixture and pores were formed by the melting of the ice. These interesting processes are now mainly of academic interest, were apparently never widely used, and have been, as Mr. Walter states, replaced by "simpler and less costly" processes.

RUDOLPH C. VALORE, JR., Materials Engineer, Structural Engineering Section, National Bureau of Standards, Washington, D. C.

*Valore, Rudolph C., Jr., "Cellular Concretes—Part 1," ACI JOURNAL, May 1954, *Proc.* V. 50, p. 773.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Bridges

British highway bridge loading

WILLIAM HENDERSON, *Proceedings*, Institution of Civil Engineers (London), Part II, V. 3, No. 2, June 1954, pp. 325-373 (including discussion)

Reviewed by ARON L. MIRSKY

New regulations now under consideration for loadings to be used in designing highway bridges are discussed. Two design loadings are suggested: for bridges subject to normal traffic, a lane loading plus a "knife-edge" (*i.e.*, concentrated) load, similar to but not identical with AASHO lane loading; for bridges subject to heavy abnormal traffic, a train of four axles, each with four wheels, simulating a modern heavy-duty trailer for hauling construction machinery or other heavy loads.

Both loadings are much heavier than those in American practice—it was noted, in the discussion, that in England there are all sorts of loads which are rejected by the railways and hence have to be transported on the highways, which is not quite the case in America.

Highway bridge slabs. Design tables for loads conforming to German Industrial Standard 1072, Highway and Rural Road Bridges, Assumed Loads (Fahrbahnplatten von Strassenbruecken. Berechnungstafeln fuer Lasten nach DIN 1072, Strassen—und Wegebruecken, Lastannahmen)

HUBERT RUSCH, *Publications* No. 106, Deutscher Ausschuss fuer Stahlbeton, E. Ernst & Sohn, Berlin, 1st Edition 1952, 152 pp.

Reviewed by ARON L. MIRSKY

The most recent revision of DIN 1075, Massive Bridges, Principles of Analysis, ntroduced the requirement that bridge slabs be designed by the theory of plates. This publication is designed to enable the busy engineer, or one who has perhaps not fully mastered the theory, to satisfy that requirement, by presenting the results of analyses of slabs in tabular form.

Part A of this handbook is a brief but adequate (for its purpose) discussion of the theory of plates, including its application to skew slabs, fixed-end and continuous slabs, and others. There are four worked examples and a 14-page index of cases, referenced to the detailed tables of Part B, which occupy 80 pages. The final two pages contain 46 references to literature, including several American papers.

The second (1953) edition includes additional material such as uniformly distributed loads.

Dams

Photoelastic experiments on the stress distribution in a diamond-head buttress dam

A. W. HENDRY, *Proceedings*, Institution of Civil Engineers (London), Part I, V. 3, No. 3, May 1954, pp. 370-396

Reviewed by ARON L. MIRSKY

The Errochty Dam of the Tummel Garry Project, Scotland, is made up of a series of diamond-head buttress units which are so constructed as to be free to act independently. Since exact analysis of the stresses in such a structure is rather difficult, model analysis was resorted to. The frozen-stress method was

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 26, No. 4, Dec. 1954, *Proceedings* V. 51. Address 18263 W. McNichols Rd., Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parenthesis following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

used to study gravitational stresses (applied by centrifuging the model to 40 gravities) and hydrostatic stresses (applied by a battery of hydraulic rams). Both two- and three-dimensional investigations of the effects of a hole in the diamond head were also undertaken. No dangerous stresses were found; the experiments also confirmed that a circular hole or large gate-shaft could be made in the head without materially increasing the stresses. Simplified calculations gave results in reasonable agreement with the experimentally-found values.

Concerning construction in the United States of America (Von Bauen in den Vereinigten Staaten von Amerika)

H. W. Puell, *Der Bauingenieur* (Berlin), V. 28, No. 12, Dec. 1953, pp. 427-430

Reviewed by Aron L. Mirsky

A German look at American dam construction practices. Author's conclusion: deserving of careful study, but not to be adopted slavishly.

Injection system of the Vrla-II dam, Yugoslavia (Sistem zaptivanja na brani Vrla II, drugi deo)

Vitomir Joksic, *Nase Gradevinarstvo* (Belgrade), V. 8, No. 3, Mar. 1954, pp. 73-81

Reviewed by J. J. Polivka

For the first time in Yugoslavia an impervious upstream blanket of asphalt concrete lining was used on a dam. After thorough investigation of soil conditions, cement grouting was recommended to prevent leakage and reduce uplift. Ninety-one injection pipes were driven to depths of 33 to 50 ft covering 18,500 sq ft and altogether 380 tons of cement were used.

Gravity dam deflections

Serge Leliavsky, *The Engineer* (London), V. 198, No. 5137, July 9, 1954, pp. 41-45

Reviewed by Aron L. Mirsky

Formula derived by author includes effects of only two factors, external water pressure (both shear and moment effects are used) and uplift, yet the formula yields results for Young's modulus, using observed deflections, which are in reasonable agreement with values calculated by other, more complicated, equations, for dams of various profiles. A method of adjusting for effect of partial reservoir filling, as in the Sollingen Dam, is also given.

Study of stresses in dams by a reduced-size model (Etude sur modele reduit des contraintes dans les barrages)

A. Nizery, G. Remenieras, and N. Beaujoint, *Annales des Ponts et Chaussees* (Paris), V. 123; No. 4, July-Aug. 1953, pp. 395-437; No. 5, Sept.-Oct. 1953, pp. 533-581

Reviewed by Aron L. Mirsky

First part discusses the theory of model analysis; considerable space is devoted to the subject of three-dimensional similitude (pp. 408-422) and to the effects of the various Poisson's ratios of the materials used for models (pp. 430-437).

Second part is concerned with experimental techniques: the various model materials (celluloid, plaster, concrete, cork, and rubber; pp. 534-552); methods of applying forces to models; and devices for measuring deformations, stresses, and displacements. A bibliography of 17 items is included.

Paper is more than just an excellent survey of the current status of model analysis of stresses in dams; it may be studied with profit by others than dam designers.

Design

Instability of equilibrium of polygonal arches (in French)

E. Giangreco, *Publication of the International Assn. for Bridge and Structural Engineering*, V. 13, 1953, pp. 125-142

Applied Mechanics Review
July 1954 (Meyerhof)

Using energy considerations and Dirichlet's theorem, author analyzes the elastic instability of polygonal arches or shells. After determining the stresses along the edges from the membrane theory, displacements along the edges are established compatible with the boundary conditions. From the second increment of the potential energy of each face, an expression is obtained for the whole structure and its minimum value gives the solution for the critical buckling load. An example of the critical load of an arch with five faces illustrates the straightforward method proposed.

Statical theory of thin shells in rectangular coordinates (Theorie de la statique des voiles minces en coordonnees rectilignes)

M. Layrangues, *Annales des Ponts et Chaussees* (Paris), V. 124, No. 1, Jan.-Feb. 1954, pp. 27-67

Reviewed by Aron L. Mirsky

In previous paper (*Annales des Ponts et Chaussees*, V. 123, No. 1, Jan.-Feb. 1953, pp. 69-105; "Current Reviews," ACI

JOURNAL, June 1953, *Proc.* V. 49, p. 965) author developed theory of thin shells using vectorial analysis and curvilinear coordinates. Current paper develops theory in rectangular coordinates; as author notes, the two methods are closely related and choice of either in practice is a matter of which is easier in a given case. Shells of several surface configurations (paraboloid of revolution, equilateral hyperbolic paraboloid, elliptic paraboloid, conoid, and third-order conoid) are investigated.

Design of concrete structures

L. C. URQUHART and C. E. O'ROURKE, 5th Edition, McGraw-Hill Book Co., New York, N. Y., 1954, 500 pp.

This standard textbook has been extensively revised, and conforms to latest codes. Appropriate as ever for a first course in reinforced concrete design, it now contains material useful for advanced courses and for practicing engineers. New material includes good treatment of wind forces, moment distribution, prestressed concrete, and the plastic theory of design. Convenient tables for practical design use are included. One omission is the subject of jointing in buildings, practice of which varies with usual designer's experience. Covers the entire field of ordinary concrete structures except specialized work such as shells or precasting.

Conditions of integrability of the tensor of total deformation in a finite transformation of a three-dimensional medium (Conditions d'integrabilite du tenseur de deformation totale dans une transformation finie d'un milieu a trois dimensions)

CHARLES PLATRIER, *Annales des Ponts et Chaussees* (Paris), V. 123, No. 6, Nov.-Dec. 1953, pp. 703-709
Reviewed by ARON L. MIRSKY

Note extending St. Venant's equations for infinitely small deformations to the case of finite deformations.

Method of characteristic coefficients for the computation of reinforced and plain members (in German)

H. SASSE, *Bautechnik-Arkiv* (Berlin), No. 7, 1953, pp. 32-61
APPLIED MECHANICS REVIEW July 1954 (Sved)

Formulas are developed expressing the axial and moment load-carrying capacities of rectangular reinforced concrete sections as functions of the dimensions of the section, the permissible (concrete) stress and characteristic coefficients; these coefficients are plotted as a function of the position of the neutral axis. Additional curves are used to locate the neutral axis for given amount and distribution of the reinforcement or, alternately, to find the reinforcement for the desired location of the neutral axis. In reviewer's opinion, the method suggested is no improvement on calculations in everyday use.

Model analysis of structures

T. M. CHARLTON, John M. Wiley & Sons, Inc., New York, N. Y., 1954, 139 pp., $5

An attempt to popularize the use of model analysis in design offices. Techniques treated require only simple tools, and apparatus are easy to use. An excellent text for college use. Abbreviated theory is developed to accompany each step. Reviews "classical" methods and presents method of "large displacements" for ease in measurement. Discusses use of MIT moment indicator and simple devices to eliminate creep in model plastics. Application of the simple "isodynamometer" devised by author is explained and described for direct determination of shear moment and axial force at an internal joint. Practicing engineers planning model tests would appreciate more photographs of apparatus, however, to amplify line drawings and descriptions.

An electrical experimental procedure for determination of bending moments of statically indeterminate frames (Ein elektrotechnisches Experimentalverfahren zur Ermittlung der Biegemomente statisch unbestimmter Stabwerke)

GR. R. KUROS, *Der Bauingenieur* (Berlin), V. 29, No. 1, Jan. 1954, pp. 10-15
Reviewed by ARON L. MIRSKY

Design of equivalent-current circuits (resistance networks) for analyzing open frameworks without sidesway. A numerical example is worked. Treatment of closed frames, and frames with sidesway, is reserved for a future paper.

Complement to the study of Vierendeel-type girders (Complement a l'etude des poutres echelles)

M. J. GRIVEAUD, *Annales des Ponts et Chaussees* (Paris), V. 123, No. 3, May-June 1953, pp. 245-280
Reviewed by ARON L. MIRSKY

Author's previous paper (*Annales des Ponts et Chaussees*, May-June 1947, pp. 311-344) is generalized to make method applicable to

analysis of any system of two members joined by two vertical posts (which need not be parallel). Three numerical examples are given in some detail, and other interesting applications are suggested.

Design of staircases

A. Krysztal, *Concrete and Constructional Engineering* (London), V. 49, No. 7, July 1954, pp. 227-234
Author's Summary

In the common method of designing staircases, it is not usual to consider that the flight-slab is supported at its intersection with the walls or by cranked beams, or to allow for the effects of thrusts of the intersections of inclined slabs. The methods described in this article take account of supports on three sides of a slab and also the support provided by the intersection of the flights with the landings.

Materials

Production of lightweight aggregates

H. G. Iverson, *Rock Products*, V. 57, No. 2, Feb. 1954, p. 110
Reviewed by Martin Mahler

Discusses the rapid growth of the production of lightweight aggregate from clays and shales, and explains the difference in weight between lightweight aggregate and ultra-lightweight aggregate, which includes materials weighing about 12 lb or less per cu ft. Although lightweight aggregate is also a cellular material, it is heavier than the ultra-lightweight aggregate, but considerably lighter than sand and gravel.

Output of gyratory crushers

K. Gauldie, *Engineering* (London), V. 177, No. 4605, Apr. 30, 1954, pp. 557-559
Reviewed by Aron L. Mirsky

Analytical investigation of gyratory crushers, which are considered as basically jaw crushers (*Engineering*, V. 176, No. 4576, 4577: Oct. 9, 1953, pp. 456-458; Oct. 16, 1953, pp. 485-486; "Current Reviews," ACI Journal, May 1954, *Proc.* V. 50, p. 815) in which a large number of elementary jaws operate in succession.

Note on the nomenclature of the calcium silicate hydrates

R. H. Bogue, *Magazine of Concrete Research* (London), No. 14, Dec. 1953, pp. 87-91
Author's Summary

Because of the ambiguity in the designation of a number of the calcium silicate hydrates, a system is proposed to overcome the confusion. Tables are presented listing the calcium silicate hydrates generally conceded to be distinct phases, together with previous designations, mineral names, method of preparation when produced synthetically, and some of the optical and crystallographical characteristics.

Importance of the different modifications of calcium sulfate for the setting time of cement (Die Bedeutung der Calciumsulfatformen bei der Abbindezeitregelung des Zementes)

G. Mussgnug, *Zement-Kalk-Gips* (Wiesbaden), V. 7, Tagungsnummer, Apr. 1954, pp. 177-185
Reviewed by H. H. Werner

The influence of the different modifications of calcium sulfate for the setting time of cement was demonstrated on old and new results. Former investigations were confirmed that gypsum (dihydrate) is the surest retarder. It is recommended to keep the grinding temperatures below 70 C to avoid setting irregularities. Setting time may also be regulated with semihydrate and artificial anhydride. Changes in setting time originate in the transformation of small amounts of alkali-oxides into carbonates and can be avoided by transforming the alkali-oxides into sulfates in the kiln.

For the evalution of the sometimes occurring thickening of the cement paste before the normal setting, the use of the Vicat apparatus with a plunger of 3-mm diameter is recommended.

Processing sand from "coral" rock

Walter B. Lenhart, *Rock Products*, V. 57, No. 6, June 1954, pp. 101-102, 115
Reviewed by Martin Mahler

A good description of this underwater process from which two sizes of sand are made, a masons sand and a concrete sand. The material quarried is classified as an oolithic limestone with coral inclusions that are sometimes quite large, hard, tough and an excellent aggregate. It is almost white in color and sand processed from the pit-run constitutes about 40 percent of the total. The remaining total is processed to produce two sizes of aggregate. Basically, the mining operation consists of stripping, then after an area has been loosened by blasting, the material is excavated by dragline and piled to drain. After a month or more, the material is loaded and hauled to the processing plant.

An excellent explanation of the screening and crushing operation is given in detail as well as the water scalping tank sizer and the dewatering units and the stacker belts leading to the storage pile.

ASTM standards on cement (with related information)

American Society for Testing Materials, Philadelphia, Pa., May 1954, 254 pp. $2.75

Prepared by ASTM Committee C-1 on Cement, this edition gives in their latest form 34 ASTM standards, including 23 test methods; 9 specifications; and 2 definitions: one for the term addition as applied to hydraulic cement, the other for the term pozzolan.

Chemical and physical test methods cover: portland cement—autoclave expansion, calcium sulfate, chemical analysis, fineness, heat of hydration, sodium oxide, and potassium oxide. Also included are test methods for: hydraulic cement—air content, chemical resistance, compressive strength, fineness, mechanical mixing, normal consistency, sampling, soundness, specific gravity, tensile strength, time of setting, and flow table. Specifications cover: cement — portland, natural, and masonry. Appendices include information on analytical balances and weights, a manual of cement testing, a list of selected references on portland cement, and an article on the principle of the methoxyl method for determining vinsol resin in portland cement.

Some observations on the conferences on pozzolanic materials and their uses (Quelques observations a propos des conferences sue les pouzzolancs et leurs applications)

A. Rio, *Revue des Materiaux de Construction* (Paris), No. 460, Jan. 1954, pp. 13-15

Reviewed by Phillip L. Melville

Conferences were held in Rome in 1953 by the Italian Chemical Society on pozzolanic materials. It was brought out that all volcanic products are not equal in behavior depending on eruptive conditions. Complex siliceous products, investigated by x-rays, result from the action of instable volcanic glasses in very fine powder. The pozzolanic materials react on the lime to form silicates, aluminates, and ferrites. New applications are foreseen for pozzolanic materials, not only with portland cement, but also with "supersulfated and aluminous" cements. The inhibiting action of pozzolanic materials on the expansive alkali-aggregate reaction is discussed as well as a number of practical applications to existing European structures.

Use of "Komplexon III" in the cement laboratory (Verwendung von Komplexon III in Zementlaboratorium)

R. Huppertsberg, *Zement-Kalk-Gips* (Wiesbaden), V. 7, No. 6, June 1954, pp. 249-251

Author's Summary

The introduction of "Komplexon" in analytical chemistry permits a quick determination of CaO, MgO, and Fe_2O_3. The author describes a method by which an analysis for a cement or a similar product can be carried out within 3 to 4 hr which (in regard to accuracy) corresponds to customary methods used hitherto. The author likewise indicates analysis prescriptions for the determination of slag sand and free lime in cement brands.

Integral waterproofing of mortar and concrete

Norman Griffiths, *Constructional Review* (Sydney), V. 26, No. 7, Nov. 1953, pp. 29-31

Discusses, in question and answer form, the nature, relative value, and effective use of integral cement waterproofers. Questions are included to help clarify some misconceptions concerning their composition, usage, and relative effectiveness. A bibliography of eight references is included.

Proportioning fine aggregates (Dosage des sables)

H. Manche, *Revue des Materiaux de Construction* (Paris), No. 461, Feb. 1954, pp. 51-53

Reviewed by Phillip L. Melville

To answer criticisms of his method of concrete proportioning published in April, May, and June, 1953, ("Current Reviews," ACI Journal, *Proc.* V. 50; Oct. 1953, p. 182; June 1954, p. 906) the author states that for the sake of economy and workability his recommendation for the use of a minimum of sand should be changed. He now concludes that fine aggregate may be increased up to 75 percent but there should not be more than ⅓ of mortar in any concrete, as the higher the proportion of larger particles the stronger will be the resulting concrete. It is also remarked that because portland cement keeps on improving in quality, a corresponding decrease in quantity is indicated.

Stability: Intrinsic curves and curves of tension and compression of a material corresponding to certain structural conditions (Stabilite, courbe intrinseque et courbes de traction et compression d'un materiau repondant a certaines conditions de structure)

M. Davin, *Annales des Ponts et Chausees* (Paris), V. 123, No. 6, Nov.-Dec. 1953, pp. 623-665

Reviewed by Aron L. Mirsky

Author first discusses the notion of "scale:" if a material is made up of components whose mechanical properties are known, and if the scale of the material and its components is such that the material appears to behave isotropically at that scale, the intrinsic stress-strain curve for the material may be deduced.

Assuming a material whose cohesive forces are entirely due to radial interactions between the centers of attraction, and whose law of attraction as a function of distance is of binomial form $[\alpha(\rho - 1) + \beta(\rho - 1)^2]$, author formulates an expression for the elastic potential from which he deduces the tension and compression characteristics and the intrinsic curves. Theoretical and experimental results are then compared for such materials as concrete and mortar. Differences are stated to be due chiefly to irreversible phenomena occurring prior to rupture.

"Elasticians" will find much of interest in this paper.

Pavements

Observations on concrete pavements (Beobachtungen an Betonfahrbahndecken)

Concrete Highway Committee of the Forschungsgesellschaft fuer das Strassenwesen e. V.; Kirschbaum Verlag, Bielefeld, Germany, 1952, 84 pp.

Reviewed by Aron L. Mirsky

A collection of 14 reports, some of them only one page long, summarizing current knowledge of various topics pertaining to concrete highways: relation between concrete streets and subsoil (Luther Schaible); protection of pavements from attack by ice-melting salts by air entrainment (Kurt Walz); investigations on air-entraining additives (H. v. Elsner); stresses and reinforcement in concrete pavements, a discussion of joints and dowels (Gustav Weil); crack formation, based on a study of a section of the Autobahn between Pforzheim and Ulm (Werner Leins); pavement jacking (Seywald, Saurler, and Willigerod); the committee's information pamphlet on maintenance of concrete streets (Herbert Eymann); experimental long pavement slabs) Gustav Weil); status of soil-cement roads in Germany (Friedrich Reinhold); "elastic concrete," a patented method of producing crackless concrete by coating part of the aggregate with a thin layer of bituminous material (Rudolf Dittrich). With the possible exception of the last, the German studies parallel American investigations; the literature cited contains a goodly proportion of American papers.

Joints sawed in pouring sequence for concrete apron

Roads and Streets, Jan. 1954, p. 37

ARBA Technical Information Digest Mar. 1954

Low joint sawing costs and virtually complete elimination of uncontrolled shrinkage cracks are among the results reported at Davis-Monthan Air Force Base, Tucson, Ariz., where a heavy-duty concrete apron, 800 x 4000 ft in area is being completed. The 15 in. thick concrete pavement is designed in 25 ft lanes which are keyed but not tied except around the periphery of the apron. The joints are being sawed by a machine which rides the forms and mounts four 12-in. saws in a single traveling frame. The method has been to cut all joints in straight 1-2-3-4 order of placing. Thus when paving was started, the joints were cut from 7 to 9 hr following, the sawing beginning in the early afternoon and ending at midnight. Blade wear is greatly reduced by the additional hardening of the concrete.

Design and construction of a high-speed test track for motor vehicles

Ralph Freeman and John A. Neill, *Proceedings*, Institution of Civil Engineers (London), Part II, V. 3, No. 2, June 1954, pp. 189-221 (including discussion)

Reviewed by Aron L. Mirsky

The flat perimeter track of disused Lindley airfield was converted to a test track on which average speeds of up to 125 mph, and a balanced speed of 84 mph, can be attained. Design and construction of this facility are described.

Airfield pavement design of the Corps of Engineers

Gayle McFadden, *Proceedings*, ASCE, Separate No. 458, V. 80, July 1954, 20 pp., $0.50

Reviews U. S. Army Corps of Engineers' methods of designing airfield pavements. Some design curves are included.

Precast concrete

Beam and pan construction in Peru

H. L. Haworth, *Civil Engineer Corps Bulletin*, U. S. Navy, V. 8, No. 7, July 1954, pp. 9-11

Describes two types of partially precast beams that in effect result in a type of joist and pan floor construction. In one type, the designed lower chord reinforcing is welded to web steel and the tops of the web steel are held by two lightweight angles bolted back to back. The lower steel is then cast into concrete having a cross section of about 2 x 6 in. The "prefabs" are fitted into the framing so that the main structural concrete girders and beams are cast in place with the pour continuing on into the joist and pan system of the floor. The actual pan consists of lightweight concrete block. Concrete is placed to about 2 in. aboye the block. After the concrete sets, the angles are removed from the web steel and reused. Protruding web steel is bent down to avoid interference with the floor finish.

The other system is essentially the same except that removable sheet metal is used as the bottom form.

Factory production of precast reinforced concrete (Werke fur Stahlbeton-Fertigteile)

O. A. Gerschberg and W. I. Soboker, VEB Verlag Technik, Berlin, 1953, 228 pp.

Reviewed by Eivind Hognestad

A translation of the Russian original published at Moscow, 1951, this book gives detailed information regarding methods used in Russia in factory mass production of precast reinforced concrete units. The book is intended for the engineering personnel of such factories and consists of 14 chapters, the contents of which are reviewed briefly below.

(1) Chapter 1 gives a general description of the nature of the precasting industry. (2) Chapter 2 concerns the common types of precast units, such as beam and slab units, wall and roof panels, columns, piles, and pipe. (3) Concrete materials and the design of concrete mixes. (4) Methods of placing concrete; vibration, vacuum, rolling of dry mixes, centrifugal spinning. (5) Accelerated curing methods; atmospheric and high-pressure steam, calcium chloride, hot water, electric curing; thermal analysis of steam curing. (6) Dynamic analysis and mechanical design of vibration tables; design of vacuum equipment; design of spinning equipment; discussion of curing pits, autoclaves, steel molds. (7) Chapter 7 contains detailed information regarding preparation of reinforcement cages, and various types of transport equipment is described. (8) Production of beams; pre-tensioning and surface vibration; compaction by rollers. (9) Production of hollow units; discussion of cores. (10) and (11) Thirty pages are devoted to various methods of mass producing slab units. (12) Production of pipe and rings. (13) Terrazzo and window sills. (14) The final chapter discusses organization and economics of precast concrete production.

In general, the book concerns production of small units only. For instance, beams over 13 in. deep are not referred to, and few spans exceed 20 ft. Containers for concrete transport are usually of 10-cu ft capacity, and cranes of 1-ton and ½-ton capacity are emphasized.

Prestressed concrete

Full-scale test on prestressed concrete beam

Engineering (London), V. 177, No. 4613, June 25, 1954, pp. 815-817

Reviewed by Aron L. Mirsky

Article, abstracted from report by P. B. Morice of the Cement and Concrete Assn., summarizes results of test to destruction of first 55-ft span I-shaped Lee-McCall beam manufactured for the 2140 ft long culvert of the Lee Conservancy Catchment Board. Load was applied at points 3 ft either side of span center, resulting in constant moment in central 6 ft length. Results indicated elastic action within design load, an ultimate moment in reasonable agreement with estimated value, factors of safety of 2.4 and 2.0 for live load and total load, respectively, and consistency of stress values obtained from deflection and strain readings.

Steel for prestressed concrete

M. W. Huggins, *Canadian Conference on Prestressed Concrete*, University Extension, University of Toronto, 1954, pp. E-1–E-20

Author's Summary

An analysis has been made of the tests and recommendations of many investigators of prestressing steel. The material is divided into two main sections: (1) What is wanted in steel for prestressed concrete? (2) What can be obtained from existing sources at reasonable cost? In conclusion some clauses for a specification are proposed.

Hollow prestressed concrete pile

Civil Engineering and Public Works Review (London), V. 49, No. 572, Feb. 1954, pp. 175-176

Reviewed by James Michalos

An English company has developed a hollow, prestressed concrete pile composed of precast units 5 ft long. The pile has a 16 in. square section with a cavity of 12 in. diameter. The pile length is made up of a head unit, an underhead unit, a toe unit, and one or more intermediate units as required.

In dealing with prestressed piling, one of the principal problems has been the extension of the pile if it is found to be too short in the driving. Provision has been made for screwing mild steel bars into the anchor blocks in the head unit. These bars can be linked in the conventional manner and the pile lengthened as required.

New type of concrete pile: the Stent-Sykes prestressed hollow pile

The Dock and Harbour Authority, Jan. 1954, pp. 34, 399, 275-276

Applied Mechanics Review
May 1954

Describes a new type of hollow concrete pile which is prestressed and sectional. Advantages are as follows: (1) Hollow piles can be made longer than solid piles of the same weight. (2) Hollow piles, being lighter for a given length, are easier to handle and cheaper to transport. (3) A sectional pile is flexible and economical, as standard units can be quickly assembled to any required length. (4) No unit weighing more than ½ ton need be handled in the factory.

Prefabricated system of prestressed concrete construction

The Engineer (London), V. 198, No. 5136, July 2, 1954, pp. 27-28

Reviewed by Aron L. Mirsky

Describes the "Intergrid" system, suitable for buildings up to four stories high. Standard precast concrete units are assembled and prestressed on site to form floor and roof systems which consist of perimeter (boundary) beams, primary beams spanning between boundary beams, and secondary beams spanning between primary beams. An unreinforced concrete slab is placed in the squares thus formed and grouted in position to form a rigid monolithic horizontal slab. Beams are actually open-web trusses, to accommodate utilities within structural depth. Columns are also precast prestressed sections with heads designed to receive beam ends of various types and with sides designed for attaching the precast cladding units.

Losses in prestressed concrete beams

Henry J. Cowan, *The Engineer* (London), V. 198, No. 5136, July 2, 1954, pp. 5-7

Reviewed by Aron L. Mirsky

Three types of loss are analyzed: precompression (elastic contraction occurring when cable load is transferred from external anchorages to concrete in pre-tensioned beams), shrinkage, and creep. Four types of beams are investigated: (1) uniformly pretensioned beams not carrying their own weight, (2) eccentrically pre-tensioned beams carrying their own weight, (3) post-tensioned beams, with the steel grouted, and (4) post-tensioned beams, cables or bars not grouted. Five numerical examples are worked to illustrate use of formulas developed and afford a comparison of the effect of the various factors.

Creep tests on prestressing steel

F. W. Gifford, *Magazine of Concrete Research* (London), No. 14, Dec. 1953, pp. 71-74

Author's Summary

Details are given of tests on 0.2-in. diameter prestressing steel maintained under approximately constant strain to determine the relaxation of stress at various initial loads, including the effect of initial overstressing. A special testing technique was developed facilitating a simple but sensitive method of measuring the force in the wires which were of a sufficient length to give a good average value of the effect of creep.

Statical indeterminacy—the foundation of prestressed concrete design

T. O. Lazarides, *Canadian Conference on Prestressed Concrete*, University Extension, University of Toronto, 1954, pp. G-1–G-15

The concept of prestressed concrete being a statically indeterminate structural material is introduced and outlined. It is defined by analogy with the concept of a statically indeterminate structure. This concept is offered as a new design tool, and the principles involved are illustrated by examples.

Report on prestressed concrete research

A. L. Parme, *Canadian Conference on Prestressed Concrete*, University Extension, University of Toronto, 1954, pp. H-1–H-9

Reviews briefly some of the recent research on prestressed concrete.

Prestressed concrete (Spannbeton)

P. Misch, *Zeitschrift VDI* (Duesseldorf), V. 96, No. 13, May 1, 1954, pp. 379-386
Reviewed by Aron L. Mirsky

Excellent survey article covering the types of prestressing (fully and partially prestressed; pre- and post-tensioned; bonded and unbonded; bars, wires, and cables; and anchorages). The various systems of end anchorages are especially well covered. Article concludes with a short discussion of some technical difficulties (friction, corrosion, and others), a few interesting applications, and a bibliography with 21 items—all German.

Significant features of prestressed concrete

G. Magnel, *Canadian Conference on Prestressed Concrete*, University Extension, University of Toronto, 1954, pp. F-1–F-37

Discusses problems and applications of prestressed concrete, including statically indeterminate structures, friction loss, ultimate load design, allowable working stresses, and others.

Properties of concrete

Freezing damage to fresh concrete (Skader ved frost paa nystoept betong)

C. J. Bernhardt, *Teknisk Ukeblad* (Oslo), V. 101, No. 31, Sept. 2, 1954, pp. 681-687
Reviewed by Eivind Hognestad

A laboratory investigation carried out at Norway's Institute of Technology is reported. Four-in. cubes were cast and subjected to a temperature of 21 or − 3 F for periods varying from 2 to 28 days. The low temperature storage was started 0 to 12 days after casting. The frozen cubes were then compared to control specimens that had not been frozen. The ratio between compressive strengths was used as a measure of frost damage. The following general conclusions are made: (1) Damage by freezing fresh concrete is related to formation of ice crystals principally on the surface of coarse aggregate particles. (2) The percentage reduction in strength was independent of the low storage temperature and nearly independent of concrete composition. (3) A strength reduction of 30 to 40 percent resulted from freezing immediately after casting. This loss was considerably reduced when the concrete was permitted to harden 8 hr before freezing. After 24 hr of hardening, the effects of freezing were small. (4) Concrete damaged by freezing does not recover even by prolonged moist curing. (5) Aluminous cements gave more favorable results, particularly if freezing was delayed until 4 hr after casting. Addition of calcium chloride (2 and 4 percent) had no effect. (6) The length of the freezing period (0 to 4 days) was not important. (7) Air entrainment did not reduce the freezing damage. (8) A few freezing and thawing tests indicated that freezing of fresh concrete does not significantly reduce durability of concretes. This part of the investigation is being continued.

Note on the observation of the setting of portland cement with the electron microscope (Note sur l'observation de la prise des ciments portland au microscope electronique)

J.-R. Robinson, *Annales des Ponts et Chaussees* (Paris), V. 123, No. 3, May-June 1953, pp. 281-286
Reviewed by Aron L. Mirsky

Study leads to conclusion that formation of minute spherical particles seen under electron microscope is due to technique employed, and does not occur under normal conditions.

Curing temperature, age, and strength of concrete

Sven G. Bergstrom, *Magazine of Concrete Research* (London), No. 14, Dec. 1953, pp. 61-66
Author's Summary

The effects of curing temperature and age on the strength of concrete can be expressed in terms of a single parameter given by the product of age and temperature, as has been suggested by Nurse and Saul. In this parameter, the temperature is reckoned from − 10 C, which seems to be a reasonable value of the lowest temperature at which any appreciable increase in strength can take place. The correctness of this assumption is verified by means of check calculations made on previously known test series and on some new Swedish tests. It is to be noted, however, that special precautions must be taken to protect the concrete from freezing during the initial period after placing, and that the humidity of the ambient air must be taken into account.

Lateral buckling of slender reinforced concrete beams

D. Vasarhelyi and Ismet Turkalp, *The Trend in Engineering at the University of Washington* (Seattle), V. 6, No. 3, July 1954, pp. 8-10
Reviewed by Aron L. Mirsky

Three beams 2 x 11 in. in section with different types of reinforcing were tested

under center-point loading. Span was 72 in., giving an *L-b* ratio of 36. Not only was lateral deflection of the beams measurable, but the fractured surfaces in all three cases were characteristic of combined bending and torsion. Reviewer agrees with authors that as sections become more and more slender, particularly as regards prestressed elements, lateral buckling of beams will constitute a problem.

Concrete for insulation (Les betons hydrauliques refractaires)

P. M. Sauzier, *Revue des Materiaux de Construction* (Paris), No. 461, Feb. 1954, pp. 31-38
Reviewed by Phillip L. Melville

Special concrete was designed to withstand the action of fire at temperatures reaching 1400 C and possibly up to 1700 C. Aggregates should be siliceous for lower temperatures and contain increasing amounts of alumina (corundum carborundum, chromite) with higher temperatures. The binder is made with high-alumina cement. Because the cement has a slightly lower melting point than the aggregate, an increase in the cement factor lowers the thermal resistance while increasing flexural and compressive strengths and vice versa. Above 1000 C there is a chemical reaction between cement and aggregates resulting in higher strengths and improved thermal properties. This concrete has practically no shrinkage and can be used 24 to 36 hr after molding or placing.

Failure of concrete in compression (La rupture du beton en compression)

Albert Joisel, *Revue des Materiaux de Construction* (Paris), No. 462, Mar. 1954, pp. 55-67
Reviewed by Phillip L. Melville

It is stated that the conflict between theory and facts of concrete failure in compression is due to the material being heterogeneous, anisotropic, and not conforming to Hooke's law at failure. It is possible to obtain breaks following the direction of compression cones, or failure plans at various angles or localized punched failures. Three principles are given to explain the breaks in pseudo-solids like concrete: (1) breaking by compressive stress starts in shear along the original isostatic plans; (2) an "elementary break" continues to spread without increase of pressure to the end of the corresponding original isostatic lines; (3) the complete failure occurs when the "specific breaking area" reaches a certain value function of the concrete and the test. The theory is applied to specimens of different shapes and sizes (reference is made to ASTM Designation C 192-47T), effect of friction, and lateral forces.

Structures

Numerous small pours complicate canal job

Contractors and Engineers, V. 51, No. 6, June 1954, pp. 36-40

Describes part of the finishing touches on the Colorado-Big Thompson project of the U. S. Bureau of Reclamation. In the 13.6 miles of the Boulder Creek Supply Canal, there are 140 structures representing approximately 600 concrete pours. The forming and placing of this concrete has proved so difficult that the average cost to the federal government is about $85 per cu yd. Since irrigation could not be interrupted, much of the concrete had to be placed during the winter.

Modern methods speed concrete work on office building

Ray Day, *Contractors and Engineers*, V. 51, No. 4, Apr. 1954, pp. 58-60

Use of mechanical shores and special runway scaffolds helped crews place concrete in this 14-story office building.

Shell dome over underground station Blackeberg (Kupoltak vid tunnelbane station Blackeberg)

S. Eggwertz, *Betong* (Stockholm), V. 38, No. 3, 1953, pp. 131-160
Reviewed by Eivind Hognestad

A shell dome spanning 69 ft over a subway station at Stockholm with a rise of 17 ft is described. Statical analysis, construction methods, and costs are discussed.

History and construction of the foundations of the Asia Insurance Building, Singapore

William J. R. Nowson, *Proceedings*, Institution of Civil Engineers (London), Part I, V. 3, No. 4, July 1954, pp. 407-456 (including discussion)
Reviewed by Aron L. Mirsky

Building, an 18-story steel-frame structure, was designed to be founded on reinforced concrete cylinders sunk through reclaimed beach. Contractor did not follow specifications, with result that adjacent ground subsided as sinking progressed. Partially sunk cylinders were successfully underpinned using precast concrete segmental lining; bearing pressures were reduced by belling out the bases.

Concrete structures for a naval base extension (in Dutch)

C. Van Leeuwen, *Cement* (Amsterdam), No. 17-18, 1954, pp. 289-295

Reviewed by John W. T. Van Erp

The existing naval base at Den Helder is being extended to about three times its present size. Construction of seawalls, piers, and bridges took place in the dry within an area temporarily walled in by an earth dam. Foundations were all on piles, using a standard concrete pile prefabricated at the jobsite. The piles were of I-section, 60 ft long, weighed 28 tons, were cast in steel forms, vibrated, and steam cured so that one pile per form per day could be produced. Compressive strength right after steam curing was 2850 psi (8-in. cube strength) for the 7 sack per cu yd mix.

Pozzolanic cement made from blast furnace slag was used, resulting in a concrete with good resistance against deterioration by sea water. The heavy piles were not driven but sunk in steel tubes, which were jetted down and the sand pumped out. The pile was then lowered and finally pushed down by a hydraulic jack which was used to give each pile a load test. The hydraulic jacks served later to pull the pipes, as soon as the piles were in their final location.

Enlargement of the "Sweden" reservoir (Utvidgning av Svedenreservoaren)

G. Evers, *Betong* (Stockholm), V. 38, No. 3, 1953, pp. 183-192

Reviewed by Eivind Hognestad

An 88,000-cu ft water tank at Stockholm was enlarged to a 350,000-cu ft capacity. A major part of the construction was carried out using slip forms.

Velsen tunnel

The Engineer (London), V. 197, No. 5133, June 11, 1954, *Supplement*, pp. xix-xxi

Reviewed by Aron L. Mirsky

Concrete tunnel under the North Sea canal at Velsen, near Ijmuiden, Holland, will handle both rail and road traffic. Road tunnel has two bores, each for two lanes of traffic, plus chambers for air supply and other services. It will be 768 m (2520 ft) long (1644 m or 5394 ft including the approaches), with a ventilation plant at each end. The single-bore railroad tunnel, to accommodate two tracks, will be 2032 m (6666 ft) long (3224 m or 10,577 ft including the approaches) and will be unventilated since the lines are electrified. Canal is meanwhile being widened and will later be deepened.

Supplement also includes interesting descriptions of the sea defenses in Holland, being rebuilt after the Jan. 31-Feb. 1, 1953, storm and tidal surge (pp. xi-xvii) and the dual-lane Amsterdam-Utrecht motorway (pp. xviii-xix). Latter is of particular interest because in addition to incorporating most of the superhighway features to which we have become accustomed, it includes cycle paths.

North-light roof with Vierendeel girders of 81-ft span

Concrete and Constructional Engineering (London), V. 49, No. 7, July 1954, pp. 221-224

A north-light roof utilized four reinforced concrete Vierendeel girders spanning 80 ft 10 in. as the main members. The girders are at 13 ft 2 in. centers and support precast rafters inclined about 45 deg. Design and construction details are given, including the "method of balancing curvatures" used for calculating bending moments on the girders.

General

Compaction of lightweight concrete by vibration (Verdichten von Leichtbeton durch Ruetteln)

Kurt Walz, *Publications* No. 108, Deutscher Ausschuss fuer Stahlbeton, W. Ernst & Sohn, Berlin, 1952, 56 pp., 8 DM

Reviewed by Aron L. Mirsky

Presents results of an investigation conducted at the Stuttgart Technical School. Variables included frequency, amplitude and duration of the vibrations imparted to the molded specimen by the vibrating table, aggregate type (crushed brick and pumice were used) and gradation, mix, and stiffness; effects of these and of air entrainment on gross specific weight, compressive strength, texture, and stability (creep strength of the fresh concrete) were noted. Comparison specimens compacted by ramming were also prepared and tested.

Building construction in the USSR

S. Balashov and V. P. Lagutenko, *Civil Engineering and Public Works Review* (London), V. 49, No. 576, June 1954, pp. 600-601

Reviewed by James Michalos

Consists of two short articles published together under the above heading. First describes the construction of the new Moscow State University. The main building is more than 32 stories high and involved the

use of new type prefabricated and semi-fabricated parts and frames. The second article discusses manufacture and use of standard prefabricated units. Over 70 percent of the residential and other domestic building now under construction are being built from these standard units.

Insulated forms cut winter concrete cost

Contractors and Engineers, V. 51, No. 4, Apr. 1954, pp. 90-92

By insulating the forms on this bridge job at Chamberlain, S. D., the contractor was able to save $3 to $7 per cu yd of concrete. Besides eliminating the fire hazard, the forms maintained more than minimum required temperature without additional heat.

Joints and construction joints (Fuger og stoebeskel)

Johannes Christensen, *Ingenioren* (Copenhagen), V. 63, No. 20, May 15, 1954, pp. 455-459

Reviewed by Rolf B. Beck

Discusses construction joints, expansion joints, and methods of joining precast elements.

Reinforced concrete floating crane (Cabria flotante de hormigon armado)

J. Cordoba Machimbarrena, *Informes de la Construccion* (Madrid), No. 61, 1954

Brief illustrated description of a reinforced concrete barge and crane boom of 50 metric ton capacity built for use in port work in the Canary Islands. Design information and reinforcement details are included.

Concrete shields—their preparation and use

Harold S. Davis, *General Electric Review*, V. 57, No. 5, Sept. 1954, pp. 43-45

Reviewed by Aron L. Mirsky

A not-too-technical article concerning the selection of aggregates for concrete for shielding nuclear reactors. Cost consciousness permeates the article.

Concrete mixing plants (Betongstationer)

H. Kullstedt, *Betong* (Stockholm), V. 38, No. 2, 1953, pp. 65-99

Reviewed by Eivind Hognestad

Advantages of mechanizing concrete production are discussed and desirable characteristics of modern mixing plants are described. The principles involved in the design of mixing plants are outlined and various practical experiences and improvements are reported. Several plants erected on the construction site in recent years are described.

Cut costs and prices with good form design

Joseph R. Proctor, *Contractors and Engineers*, V. 51, No. 5, May 1954, pp. 24-28

A general description of considerations necessary for economical design and use of forms.

Mechanical strain gage

Engineering (London), V. 177, No. 4612, June 18, 1954, p. 791

Reviewed by Aron L. Mirsky

Brief description of inexpensive Whittemore-like gage designed for taking readings in succession at large number of points on a concrete structure rapidly (200-400 per hr) and with reasonable (± 12 psi) accuracy.

High-pressure steam used for 44 years

Concrete, V. 61, No. 9, Sept. 1953, pp. 18, 20, 22, 33

Describes operations in a concrete block plant, with special emphasis on high-pressure steam curing set-up.

Pitfalls in flat-plate construction and how to avoid them

Stephen D. Teetor, *Engineering News-Record*, V. 152, No. 11, Mar. 18, 1954, pp. 49-50, 52

Reviewed by S. J. Chamberlin

Stress analysis requires an experienced, capable engineer backed by an expert checker. Empirical coefficients may lead to dangerous errors. Top and bottom bars should have sufficient lap and alternate bottom bars should be extended into the columns or next panel, or bent up. Actual heights of top bars need to be carefully checked in the field for thin slabs. Some methods of saving steel are questionable since they complicate placement of reinforcement and require more vigilant field control. Field inspection should be in charge of an engineer who is both qualified as a designer and versed in field work, and has a sufficient number of inspectors for details. Concrete control requires an inspector at the batching plant and it is suggested that the ready-mixed concrete companies insist on accepting only sealed, tested shipments of cement. Cylinders should be taken, handled, and tested by competent parties under the control of the engineer. When forms are stripped, the slab should be reshored to the floor below. Often, two stories should be reshored for adequate support of loads above. Suggestions are made for the improvement of municipal building codes.